DEVELOPMENT OF A STORMWATER RUNOFF MODEL

FOR THE TULSA AREA

By

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER I

INTRODUCTION

Land is our most fundamental resource. Until recently, land has been traditionally viewed as "private property" which owners may use as they please. Air and water, on the other hand, have been viewed as resources which are shared by all people. Pollution of the air and water has been recognized as harmful to the general public. The public has now become aware that land use, and its development, have an effect on the general public (1).

Major land alterations have taken place in the urban environment, which have brought about an increasing awareness of public harm associated with different land uses, thus diminishing the traditional view of land as being "private property." The public has become increasingly interested in providing regulations, through an appropriate agency, which guide the use of such land development.

Tulsa began a period of rapid growth during the period between 1950 and 1980. Between 1950 and 1970, Tulsa had grown from a city of approximately 50 square miles to nearly 170 square miles. The population had increased from 251,686 to 330,350 people (2).

The demand for land also grew during this expansion period. The growth created a demand to develop vacant land into real estate for residential and commercial uses. Sufficient consideration was not always given to the impact of this development.

Tulsa has a history of flooding. In 1959, Tulsa experienced flooding over much of the city. Mingo, Joe, and Little Joe Creeks each flooded after receiving 3.24 inches of overnight rainfall.

In 1965, over 100 square miles of land was annexed into the city limits. The city received two watersheds with that annexation--the Mingo and Joe Creek watersheds. Along with those watersheds came the flooding problems associated with the urbanization occurring within them.

Tulsa continued to have floods: June 25, 1968, and May 30, 1970 are dates of historical flooding. On June 30, 1974, a new flood record on three watersheds was reported. On May 30, 1976, three people were killed during a flood that caused $35 million in damages (3).

By the mid 1970's, the public outcry for regulatory support had reached a new high. The citizens were requesting that land development be regulated to provide flood protection. The impact of urbanization without regard to flooding had been realized.

In October, 1975, the City of Tulsa responded to the problem. The Board of Commissioners of the City of Tulsa adopted the first of several flood plain building moratorium ordinances. Although this measure was a short-term solution, it permitted a period of time to study the problem and alleviated the potential of additional flooding areas.

In 1976, a hydrology section was established in the City of Tulsa's Engineering Department to aid in the regulation of flood plain development and drainage problems. On May 16, 1976, the Board of Commissioners adopted 16 regulations for the design of stormwater runoff systems and detention facilities on all new land developments. The most important regulation, detention facilities on all new land developments, required that all residential developments of ten acres or more in size--and all other types of development of two acres or more in size--provide stormwater detention storage to account for the increased runoff resulting from urbanization. The City of Tulsa had become the regulatory agency assessed with the responsibility of regulating the development of land with respect to flood control.

There were few hydrograph models available to determine the impact of urbanization on small watersheds. The model used most often was Snyder's (4) synthetic unit hydrograph model in the HEC-1 computer model. Because some developments were densely populated and other were not, watershed characteristics were modified to consider these differences in urbanization.

This thesis presents the modification of the Soil Conservation Service runoff curve number method to be used in a hydrograph procedure for small watersheds. The purpose of the modification is to provide a quick and easy method to aid in the regulation and design of developments within the urban environment.

CHAPTER II

LITERATURE REVIEW

In an urban environment, natural stream channels that collect and carry runoff waters are often replaced by artificial drainage patterns. These man-made drainage networks behave quite differently than the watershed did in its natural state. Because of this difference, the analysis of the urban watershed is unlike that of the rural watershed. Literature concerning urban watersheds in the Tulsa area will briefly be reviewed.

The design criteria manual for the City of Tulsa (10) allows two types of hydrograph methods: Snyder's synthetic unit hydrograph (4) and the Soil Conservation Service (SCS) (8). Although the design criteria manual only specifies these two methods, several others will be reviewed as they apply to urban watersheds.

The most used method for determining the peak runoff rate for small watersheds is the rational method. Most of the existing storm drainage systems have been designed with this empirical formula. Although the method is not a hydrograph method, it is still used for determination of peak runoff rates for small watersheds.

A handbook for the design of small dams (5) presents a modified rational method as a hydrograph approach. It presently is not used in the Tulsa area for design but is being tested on watersheds for comparison with other methods.

The primary hydrograph method used in the Tulsa area is Snyder's synthetic unit hydrograph. This is mainly due to the influence of the Tulsa District office of the Corps of Engineers, which relies solely on the Snyder method for the design of water resource projects within the United States.

Snyder's method relies upon correlation of the dependent variables of lag time and peak discharge with various physiographic watershed characteristics. The Snyder method was developed in the Appalachian Mountain region and has been extended to model many other watersheds.

Snyder's unit hydrograph coefficients change considerably with the stages of urban development. The Tulsa District office of the Corps of Engineers has tabulated Snyder's coefficients to use in the Tulsa area for the impact of urbanization. The input parameter for modeling the effect of urbanization is simply the percent of urbanization. The parameter is not easily defined in those terms, since it is a rather judgmental guideline and will vary from person to person.

Beard developed an urban runoff model for Tulsa, Oklahoma, in August of 1978 (6). Beard determined that the basin lag correlated best with the drainage area size but not with stream length or degree of imperviousness. Beard relates the parameter's time of concentration (T_c) and the attenuation constant (R) to the basin size.

The other method allowed by the design criteria manual is the Soil Conservation Service Hydrograph. The wording of the phrase, the "Snyder Synthetic unit hydrograph method or the Soil Conservation Service derivative thereof, shall be used for the design of all detention facilities,"[1] had significant meaning. The Soil Conservation Service did not have a

[1] Design Criteria Manual, Sec. 2.11, A, 4, Tulsa Engineering Department, 1978, p. 22.

hydrograph model for small watersheds, but the triangular unit hydrograph was believed to be a derivative of the Snyder unit hydrograph.

The Soil Conservation Service published Technical Release No. 55 (TR55) in January of 1975, which was titled "Urban Hydrology for Small Watersheds" (7). The publication presented a curve number approach to the modeling of a small urbanized watershed. The method was not a hydrograph procedure but consisted of a tabular and graphical approach.

A Tulsa engineer, Fred Gauger, first applied the SCS unit hydrograph with the curve number method presented in TR55. Gauger's hydrograph appraoch was the first to establish a curve number hydrograph model for small watersheds in the Tulsa area.

The Soil Conservation Service has a hydrograph model using the curve number approach for watersheds larger than 2000 acres. It uses the curve number method to arrive at the runoff hydrographs. This model will be discussed in detail later.

The development of the Tulsa Area Runoff model (TARM) requires an in-depth investigation of the modifications needed of the Soil Conservation Service curve number approach to develop a runoff model for small watersheds. Each segment of the model is explained in detail.

CHAPTER III

DEVELOPMENT OF THE RUNOFF MODEL

3.1 Runoff Model Parameters

As stated earlier, a hydrograph runoff model was needed to estimate the increase in runoff cuased by urbanization. This model could then be used to size the volume of an on-site detention facility required to offset the increase in runoff caused by urbanization. The input parameters to be used in the model were those used by the Soil Conservation Service (SCS) curve number method.

Urbanization can change watershed runoff characteristics in many ways. The construction of a road or building causes a once permeable surface to become impermeable. Rainfall that normally recharges the groundwater by infiltration now becomes runoff. This process of development causes an increase in the volume of runoff for a given rainfall.

Another change of watershed characteristics caused by urbanization is the interception of flow paths. Watersheds in their natural conditions will generally develop a dendritic type of drainage pattern. Because of the natural overland flow characteristics, the velocity of the runoff is usually slow and the distance it travels to the mouth of the stream is long. When development occurs, the natural drainage pattern is intercepted by man-made channels that have much more conveyance. Streets, driveways, and storm sewer systems are examples of man-made channels with a greater capacity to convey water. The runoff is carried much faster and travels a shorter distance to the mouth of the system. The net effect is that the total runoff takes place in a shorter time period with increased velocities. Both factors combine to cause a much greater peak in the runoff hydrograph.

The SCS curve number method covered all the basic parameters that were needed in the development of a runoff model for small watersheds. The volume parameter is computed using soil type, cover, and land use as variables. The runoff-time relationship is defined by the slope of the watershed, flow length of the natural channel, and a surface retardance factor.

A runoff model consists of several different parts of segments that make up the complete model. In the development of the Tulsa Area Runoff Model (TARM), each segment was studied separately to determine whether it was applicable to the model. The three segments of the model are the synthetic unit hydrograph, the design rainfall pattern, and the abstraction procedure. The latter two segments were modified from the original SCS development for use in the TARM.

3.2 The Unit Hydrograph

The relationship of runoff rate versus time for a watershed basin is known as a hydrograph. It is possible to obtain such a hydrograph directly from the flow records of a gaged stream. This "natural hydrograph" can only be obtained for existing conditions and cannot be used for determining the impact of urbanization. The data collection procedure needed to develop the natural hydrograph is both expensive and time consuming. Therefore, many empirical mathematical relationships have been developed.

The empirical models started with the rational method of determining the peak rate of runoff. The rational method was developed in the 19th century and is still used today. However, it is not a hydrograph model and cannot be used to determine the volume of runoff.

A hydrograph similar to the natural hydrograph is the "unit hydrograph." The unit hydrograph is the runoff rate versus time relationship that would occur if a unit amount of rainfall (one inch) were to fall in a specified period of time.

Another type of hydrograph is the synthetic hydrograph. A synthetic hydrograph is an empirical hydrograph based on runoff characteristics of the watershed. Because it is a mathematical model, it can be applied quickly to determine the change in runoff due to changes in a watershed.

In 1932, L. K. Sherman advanced the theory of the unit hydrograph. Using the fundamental principles of superposition, the unit hydrograph becomes a flexible tool for developing a synthetic hydrograph. The unit hydrograph used in the TARM is a dimensionless unit hydrograph developed by Mockus (8). The dimensionless units of this hydrograph enable it to be used for any size basin where the hydrograph parameters are determined.

The SCS Technical Release No. 55 presents a curve number method for determining the time of concentration, travel time, and lag parameters to be used in the dimensionless unit hydrograph. Two methods are presented: the hydrograph method and the modified curve number method. Both methods can be used in this model, but the author prefers the hydrograph or time of concentration method.

The hydrograph method as described in Technical Release No. 55 (TR-55) consists of determining the overland flow times and the corresponding travel time in the channel to the mouth of the basin. This becomes the time of concentration of the basin. The time of concentration is defined as the total travel time for runoff to proceed from the uppermost portion of the basin to the point of discharge in the basin. A simple relationship of basin lag time to the time of concentration is assumed to hold true. This relationship is:

$$Lag = 0.6\ T_c \qquad (1)$$

where Lag is lag time (from center of excess rainfall to peak of the unit graph) in hours, and T_c is time of concentration.

The lag parameter is then used in the dimensionless unit hydrograph. The relationship presented in Equation (1) was found to be true for most urbanized watersheds by the SCS.

The second method is the modified curve number method. The curve number method was originally developed for agricultural watersheds, and was later modified to model the effects of urbanization (9). The lag parameter used in the curve number method is an empirical relationship based on the hydraulic length of the watershed, a flow retardance factor, and the average watershed land slope. The curve number lag equation is:

$$Lag = \frac{l^{0.8}\ (S+1)^{0.7}}{1900\ (Y)^{0.5}} \qquad (2)$$

where

Lag = lag time (from center of excess rainfall to peak of the unit graph), in hours;

l = hydraulic length of the watershed, in feet;

S = potential abstraction, in inches; and

Y = average watershed land slope, in percent.

The potential abstraction is defined as:

$$S = \frac{1000}{CN'} - 10 \qquad (3)$$

where CN' is the flow retardance factor. For most cases it is equal to the curve number (CN).

Caution must be given in the determination of the average watershed land slope. Engineers have a tendency to use the stream slope of the basin or the weighted stream slope because many of the other hydrograph models use such parameters. On large basins the lag time is determined primarily by the travel time in the stream of the basin. In that case the stream slope or weighted stream slope is important to the model.

On small basins the predominant travel time is the initial travel time of the runoff to the point where it meets the stream. It is the overland flow time to the stream. Thus the upper slope of the basin is used to model this sheet flow.

The lag computed by Equation (2) must be modified before it is used in the dimensionless unit hydrograph to account for the nonhomogeneousness of the urbanized basin. If the basin is left in its natural state, Equation (2) will accurately model the basin lag. As it is developed, a watershed loses its homogeneous characteristics and becomes a mixture of pervious and impervious soil cover.

Equation (2) assumes that the entire basin has a uniform cover of a single soil type with a corresponding single retardance factor. In practice, the curve number assigned is a weighted curve number based on types of use and types of soil in the watershed. The lag computed by Equation (2) must be modified by Equation (4). The lag modifiers can be determined by use of Figure 1.

$$\text{Lag} = \left[\frac{l^{0.8}\ (S+1)^{0.7}}{1900\ (Y)^{0.5}}\right] (\text{LMI})(\text{LMC}) \tag{4}$$

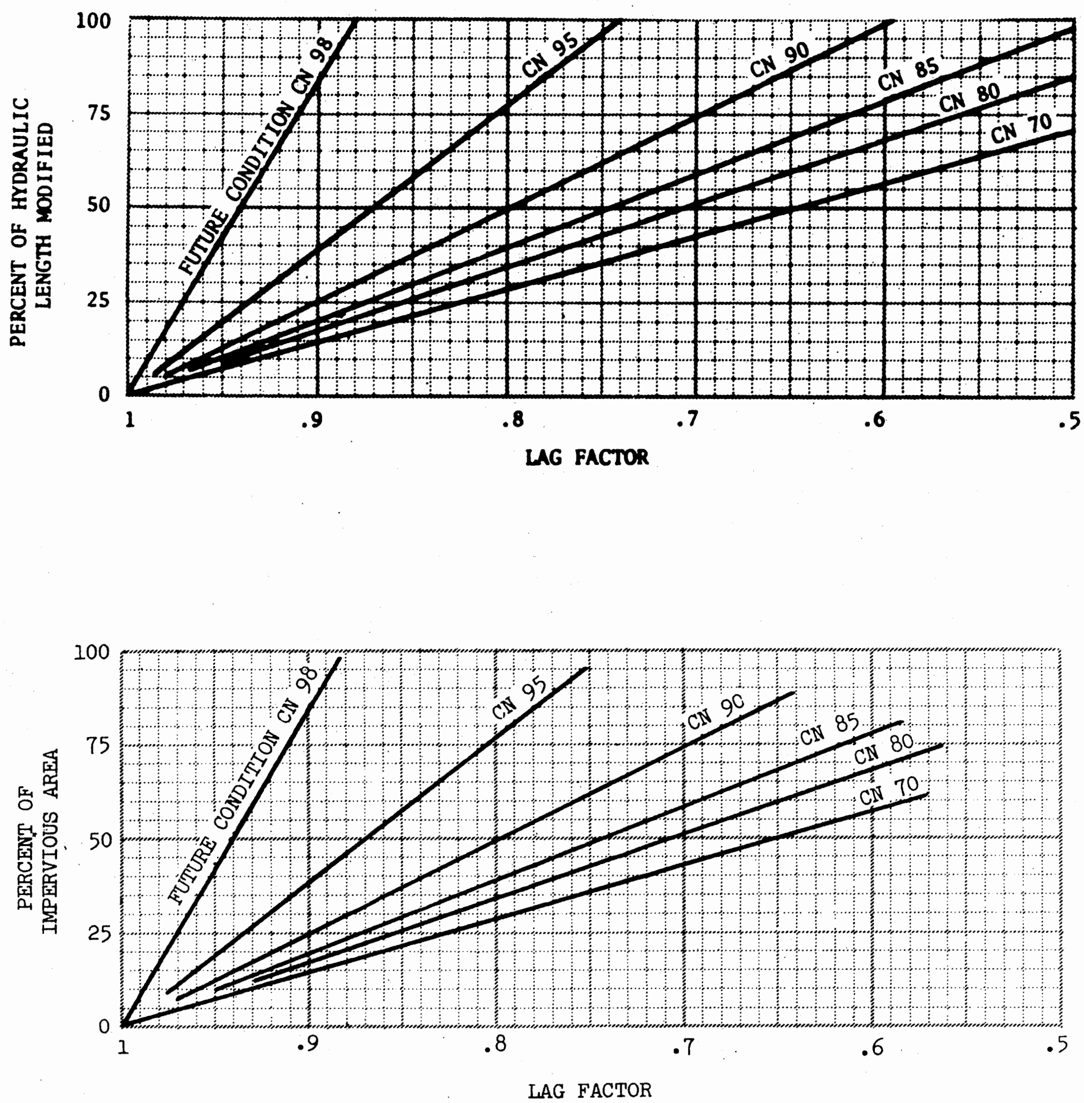

Figure 1. SCS Lag Modifiers

where LMI is lag modification due to percent imperviousness, and LMC is lag modification due to percent of hydraulic length modified.

Both methods of determining the basin lag, the hydrograph or time of concentration method, or the modified curve number method have distinct advantages and disadvantages. The hydrograph method is useful because engineers are trained to think in terms of the time of concentration. The time of concentration is used in the rational method, which is used to design storm sewer systems. Thus the time of concentration is already available in many instances because the storm sewer system has already been designed. Its only disadvantage is that the modified curve number must be determined, because it is used in the abstraction procedure described later.

The modified curve number method of determining the lag time is useful for initial calculations to determine the feasibility of a project. The curve number is easily determined for most developments. Table I gives runoff curve numbers for selected agricultural, suburban, and urban land uses. The disadvantage to determining the lag by Equation (2) is that it sometimes gives a false value for the lag time. The nonhomogeneous characteristics of the watershed are not always accurately represented by the lag modifier presented by Equation (4).

The curve number that is used in Equation (3) is determined by use of Table I or by a weighted procedure described in TR 55. The curve number is a measure of the soil type and land use factor that is used in Equation (2) and in the abstraction procedure defined later.

The computed lag is one of the parameters used in the dimensionless unit hydrograph. The other parameter used is the peak runoff rate (q_p) that is computed by Equation (5):

TABLE I

SCS RUNOFF CURVE NUMBERS

Land Use Description		Hydrologic Soil Group A	B	C	D
Cultivated Land:[1]	without conservation treatment	72	81	88	91
	with conservation treatment	62	71	78	81
Pasture or Range Land:	poor condition	68	79	86	89
	good condition	39	61	74	80
Meadow:	good condition	30	58	71	78
Wood or Forest Land:	thin stand, poor cover, no mulch	45	66	77	83
	good cover[2]	25	55	70	77
Open spaces, lawns, parks, golf courses, cemeteries, etc.					
good condition:	grass cover on 75% or more of the area	39	61	74	80
fair condition:	grass cover on 50% to 75% of the area	49	69	79	84
Commercial and business areas (85% impervious)		89	92	94	95
Industrial Districts (72% impervious)		81	88	91	93
Residential:[3]					
Average Lot Size	Average % Impervious[4]				
1/8 acre or less	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
Paved parking lots, roofs, driveways, etc.[5]		98	98	98	98
Streets and Roads:					
paved with curbs and storm sewers[5]		98	98	98	98
gravel		76	85	89	91
dirt		72	82	87	89

[1]For a more detailed description of agricultural land use curve numbers, refer to National Engineering Handbook, Section 4, Hydrology, Chapter 3, August, 1972.

[2]Good cover is protected from grazing and litter and brush cover soil.

[3]Curve numbers are computed assuming the runoff from the house and driveway is directed towards the street with a minimum of roof water directed to lawns where additional infiltration could occur.

[4]The remaining pervious areas (lawn) are considered to be in good pasture condition for these curve numbers.

[5]In some warmer climates of the country a curve number of 95 may be used.

$$q_p = \frac{484\ A\ Q}{T_p} \tag{5}$$

where

q_p = peak runoff rate, cubic feet per second;

A = drainage area, square miles;

Q = excess runoff, inches (one inch for unit hydrographs); and

T_p = time to peak, hours.

Figure 2 presents the dimensionless unit hydrograph developed by the SCS. Both the triangular and the curvilinear unit hydrograph have 37.5 percent of the total volume on the rising side of the hydrograph. The dimensionless form is centered around the time to peak, (T_p), which has one unit of time with one unit of peak discharge, (q_p).

3.3 The Rainfall Pattern

Drainage structures are usually designed for a given frequency of flood. Bridges are often designed to pass a 50-year flood frequency with one foot of freeboard, whereas a box culvert may be designed to pass only a 25-year flood frequency under a head of several feet. The frequency of a flood is established by a statistical regression analysis requiring many years of flow data. With years of flow data on a watershed basin that does not change its runoff characteristics, the frequency of a flood can be accurately determined by statistical analysis.

Watershed characteristics do change because of the impact of urbanization which in turn has an effect on the frequency of the floods. Urbanization usually increases the peak runoff rate which tends to increase the frequency of flooding. A common expression used among hydrologists is that today's 100-year flood will become tomorrow's 50-year flood.

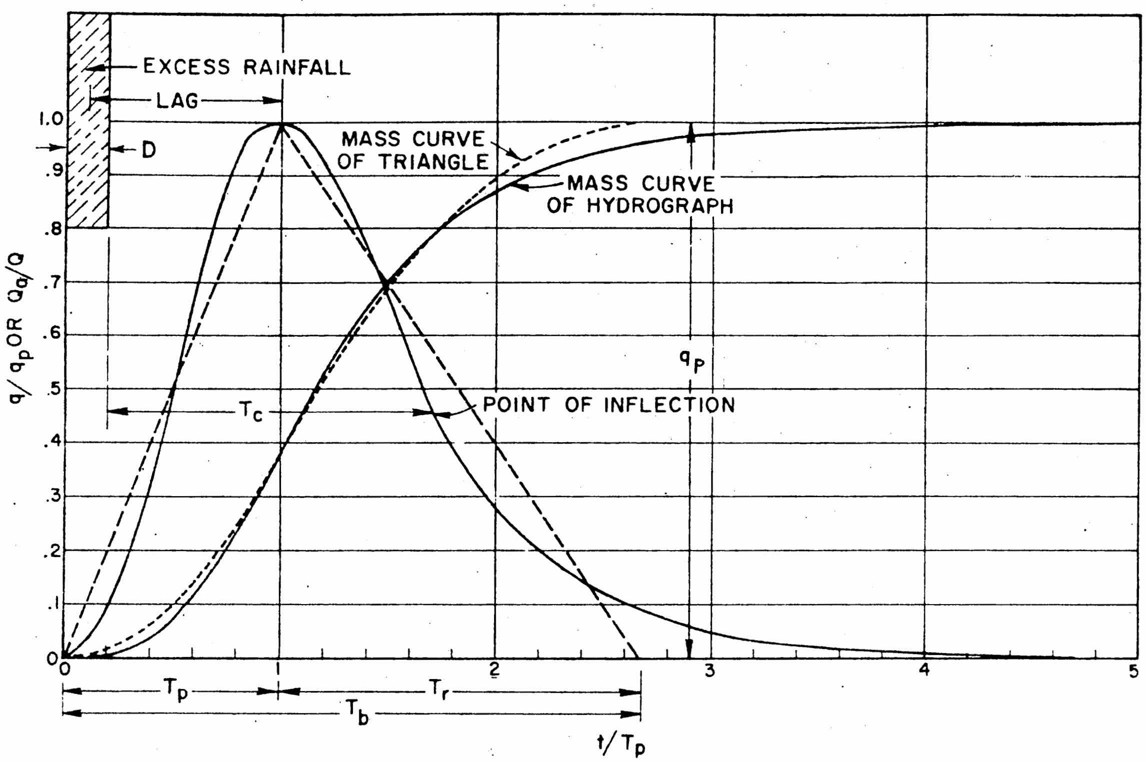

Figure 2. SCS Dimensionless Curvilinear Unit Hydrograph and Equivalent Triangular Hydrograph

Since watershed basins change with time and hydraulic structures are required to be designed to provide protection to a given frequency storm, stream flow gaging cannot be used for the design. Some other method must be used if stream flow records are inadequate or if the watershed is being urbanized.

Statistical rainfall data are then used in models on the assumption that a 25-year rainfall will produce a 25-year flood. This assumption appears to be a reliable method, but in fact is not true. The duration and intensity of the rainfall within a storm will cause different runoff rates. Two six-hour storms of the same frequency will produce different runoff rates due the dispersion of the rainfall within a storm. However, if the procedure is used with caution, the method can perform adequately for most design conditions.

3.3.1 The Balanced Rainfall Pattern

At the beginning of the development of the TARM, the rainfall pattern used in the Tulsa area was one developed by the Corps of Engineers, called the balanced rainfall pattern. This method was developed by Leo Beard.[2]

The balanced rainfall pattern consists of taking the mass rainfall for the first 15-minute period of a given frequency storm and placing it past the middle of the design storm. Then the next highest 15-minute period of the mass rainfall is placed in front of the highest 15-minute increment. The third highest 15-minute period is then placed after the highest and the process continues in this back-and-forth process. An

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[2] Conversation with Beard during a HEC-1 Training Seminar, March, 1980.

example would be the 15-minute period mass rainfall curve that fell by the series 1-2-3-4-5-6-7-8 in decreasing amounts of rainfall. The mass rainfall would then be rearranged in a 8-6-4-2-1-3-5-7 pattern. If a six-hour storm with a 15-minute increment were used, the mass rainfall curve would fall in the same 1-2-3-4-5-. . .-20-21-22-23-24 pattern. The six-hour storm would be arranged in a 24-22-20-18-. . .-2-1-3-5-. . .-19-21-23 pattern. The middle two hours of the six-hour storm is the same pattern as the two-hour storm. The six-hour storm would have two hours of rainfall occurring before and after the same two-hour storm.

This type of storm pattern was developed because of the belief that since it follows the statistical mass rainfall for a given frequency storm, it would then be statistically correct for any duration storm. For any frequency of storm, any duration storm would give the statistically correct amount of rainfall at any interduration within the storm.

This balanced rainfall pattern was used in the early model of the TARM. The pattern caused a problem in that if a longer duration storm were used, a larger peak runoff rate was produced. This problem developed regardless of the size or type of watershed basin selected. This problem was even compounded by use of the SCS abstraction pattern presented later.

After discussing the problem of using the balanced rainfall pattern with engineers around the Austin, Texas, area and with engineers in the City of Dallas, it became apparent that another type of design storm pattern would have to be developed.

3.3.2 The Severity Ratio

If a storm were plotted in a dimensionless form, such as the dimensionless unit hydrograph presented earlier, the degree of severity of the storm will be shown. From this dimensionless plot of the storm, the author has developed what is called a severity ratio. The severity ratio is the percentage of the total rainfall that fell during the intense portion of the rainfall divided by the percentage of the total time in which the intense rainfall occurred. This severity ratio is presented in Equation (6):

$$\text{Severity Ratio} = \frac{\text{\% of total rainfall for most intense portion}}{\text{\% of total storm duration of most intense portion}} \tag{6}$$

In Figure 3 the balanced rainfall pattern is plotted in dimensionless form for four different duration storms, for 15-minute time increments. Table II tabulates the same four storms and quantifies the information.

It is important to notice that all the balanced rainfall storms have the same peak intensity rates: 7.40 inches per hour or 1.85 inches for 15 minutes. This intensity rate is for the Tulsa area. What happens is that the longer duration storms simply build upon the shorter duration storms. Although the balanced rainfall pattern may be statistically correct in its format, it does not work well for comparing different duration storms for small watersheds. It is a single storm that was developed by a series of individual events.

3.3.3 The SCS Type II Rainfall Pattern

The rainfall pattern that was investigated next in the development of the Tulsa Area runoff model was the SCS type II rainfall pattern. The type II pattern is a spring and fall frontal type of storm that is common

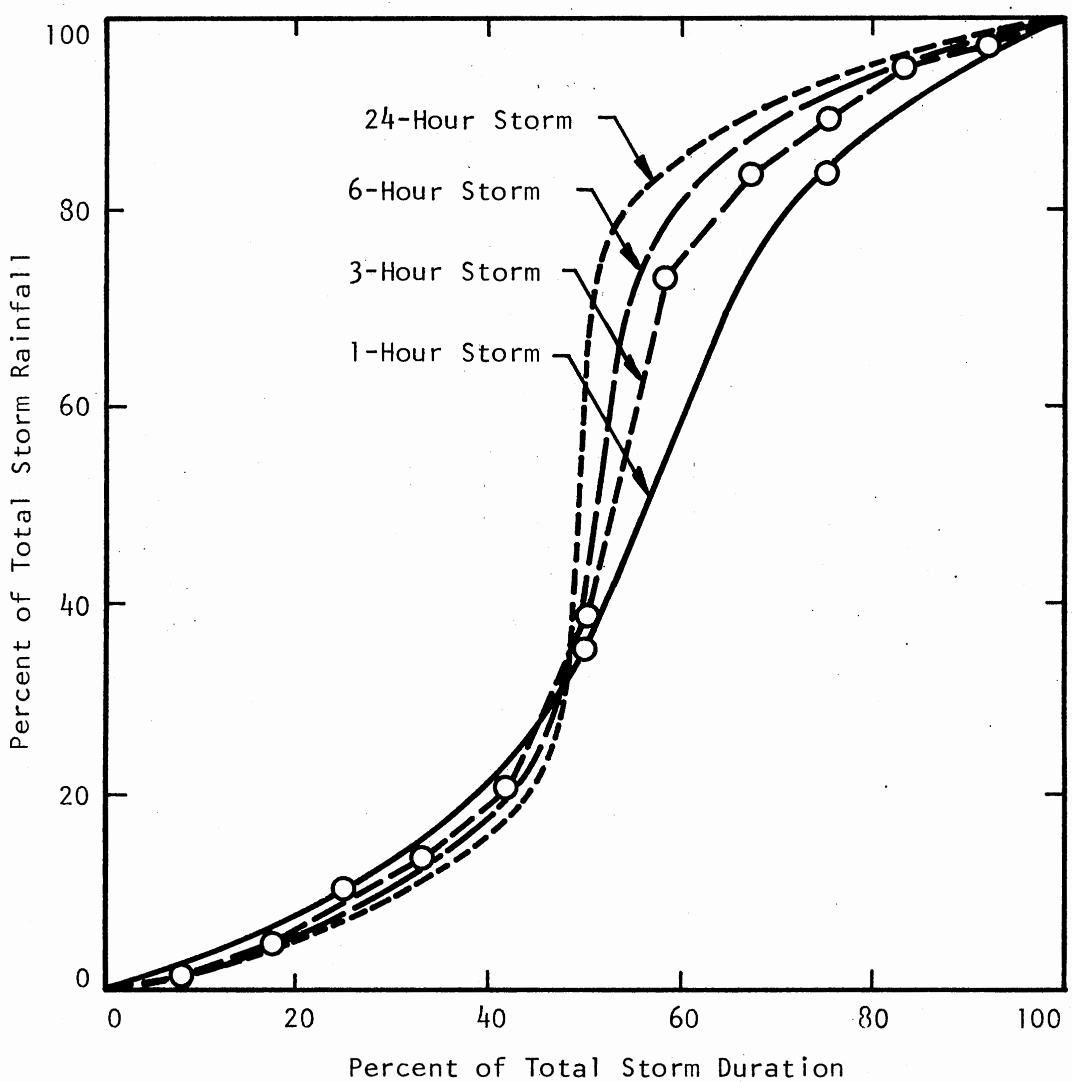

Figure 3. Balanced Rainfall Pattern

TABLE II

BALANCED RAINFALL PATTERN

Storm Duration	Total Rain-fall, in.	Peak Intensity Rate	Duration of Peak Intensity	Severity Ratio
1 hr	3.79	7.40 in./hr	1.85 in./15 min	1.95
2 hr	4.86	7.40 in./hr	1.85 in./15 min	3.05
3 hr	5.40	7.40 in./hr	1.85 in./15 min	4.11
4 hr	5.74	7.40 in./hr	1.85 in./15 min	5.16
5 hr	6.06	7.40 in./hr	1.85 in./15 min	6.11
6 hr	6.45	7.40 in./hr	1.85 in./15 min	6.88
24 hr	8.80	7.40 in./hr	1.85 in./15 min	20.20

$$\text{Severity Ratio} = \frac{\text{\% of total rainfall for most intense portion}}{\text{\% of total storm duration of most intense portion}}$$

to the central midwestern region of the United States. The SCS has four different types of storms; however, only the type II storm is applicable to the Tulsa area. The SCS only recognized two durations of storms, the 24-hour type II and the 6-hour type II rainfall pattern storms. The 6-hour type II storm is presented in Figure 4.

The SCS 6-hour type II storm was put in dimensionless form and compared to actual storms that had caused flooding in the recent history of Tulsa. The rainfall storms in the Tulsa area are presented in the Appendix. The severity ratio was computed for each of the local storms and is presented in Table III.

In Table III, two items must be given attention. First, no reference is made to the frequency of the rainfall. By plotting it in a dimensionless form, only the distribution of the rainfall within the total duration of the storm is plotted. The May 30, 1976 storm at gage 5 is in excess of a 100-year storm for a 3½-hour duration rainfall. Yet it has a severity ratio of 2.4, which is less than the May 9, 1970 storm at gage 5, which had a severity ratio of 2.6. The May 9, 1970 storm had a 10-year frequency rating for a 4-hour, 15-minute duration. The severity ratio is an indication of the interdistribution of the rainfall and is not dependent on the frequency of the storm.

The second item is that most of the storms that have produced flooding in the Tulsa area have all been of six hours or less in duration. The six storms in Table III ranged from 2½ hours to 5½ hours. This led to the investigation of modifying the SCS type II six-hour storm, so that it could be used for durations between one and six hours.

Technical Paper No. 40 (11) shows that there is a greater probability of receiving an intense one-hour storm of a given frequency than of

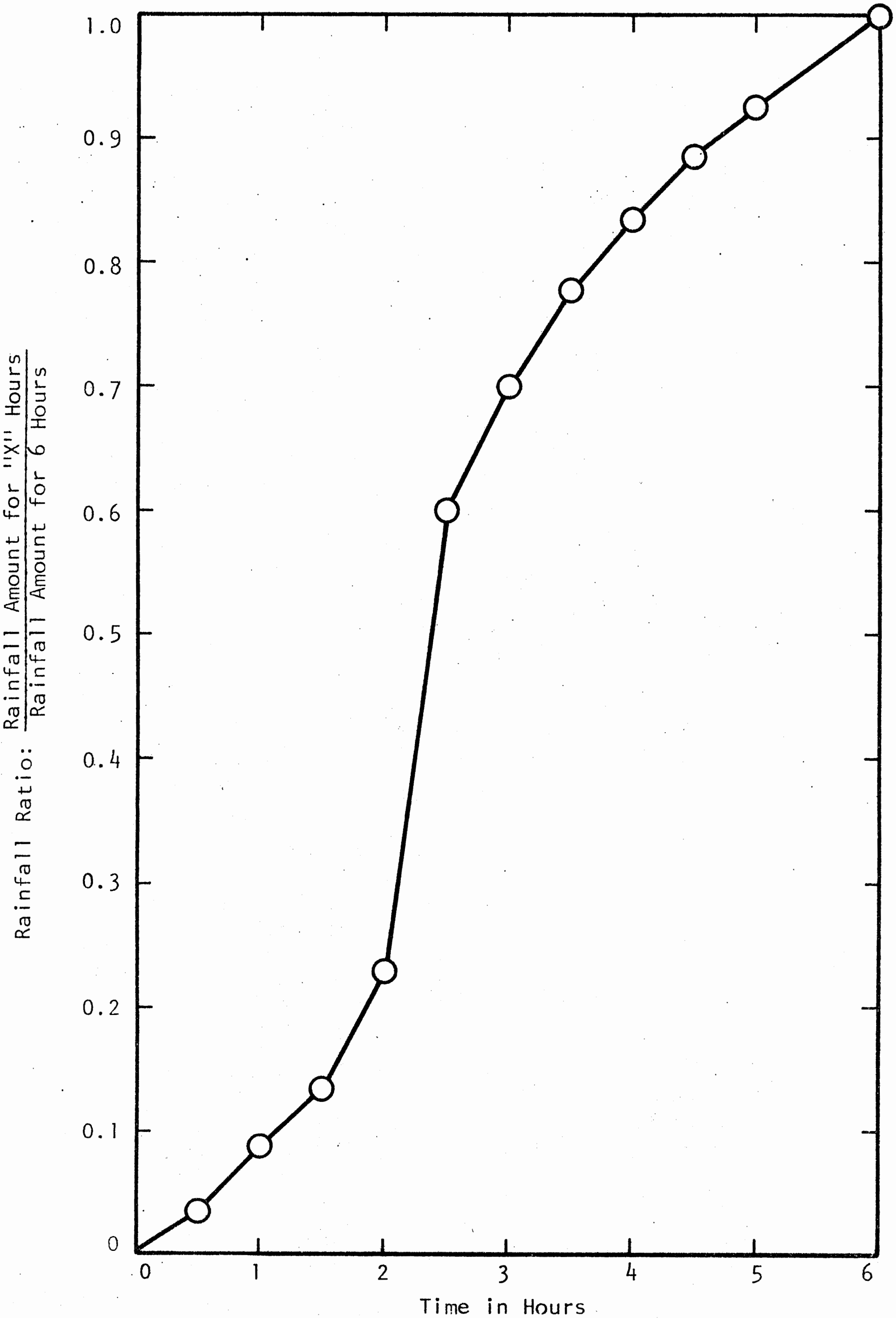

Figure 4. SCS 6-Hour Type II Rainfall Pattern

TABLE III

TYPICAL STORM INTENSITIES IN TULSA, OKLAHOMA

Date and Location	Duration of Storm	Total Rainfall, in.	Severity Ratio
1. May 9, 1970 Gage #5	4 hr-15 min	5.55	2.6
2. May 30, 1976 Gage #3	4 hr	5.35	2.4
3. May 30, 1976 Gage #5	3 hr-30 min	7.15	2.0
4. June 23, 1979 Gage #13	2 hr-30 min	5.15	1.9
5. July 6, 1979 Gage #14	5 hr-30 min	5.77	2.2
6. June 20, 1979 Gage #12	3 hr-15 min	5.85	2.3

receiving a six-hour storm of the same frequency. A one-hour storm also generally had a greater chance of occurring in any given month than the same frequency 24-hour storm.

The SCS type II six-hour storm has a severity ratio of 4.11. The storms presented in Table III have an average severity ratio of 2.2. The SCS type II storm was chosen to be modified for three reasons, even though it is indicated to have a greater severity than those occurring in the Tulsa area. The reasons for selecting the SCS type II storm are:

1. The TARM was intended to be used for small watersheds in any location, not just the Tulsa area.

2. The rainfall presented in Table III and other Tulsa area storms are not conclusive enough to warrant another type of storm pattern.

3. The TARM was intended to use the SCS curve number input parameters. The intent was to use as much of the SCS information as possible so that the model would not be a mixture of several different models.

The SCS type II six-hour storm was modified for three reasons: They are:

1. There is a greater probability of intense short duration storms than of longer duration intense storms of the same frequency.

2. The Tulsa area has a history of flooding by short duration storms. There have been few 24-hour duration storms that have caused flooding in comparison to short duration storms.

3. By using short duration storms, the TARM could be adapted for use on small capacity computers. Presently the TARM is being used on a Texas Instrument TI-59 programmable calculator.

3.3.4 The Modified SCS Type II Rainfall Pattern

The design storm that is presently being used is the Modified SCS type II pattern. It is the SCS type II six-hour storm put into a dimensionless form so that it may be used for storm durations between one and six hours. The severity ratio of the storm is 4.11, which is the same as the six-hour type II storm. The intensity ratio at the peak of the storm is greater for short duration storms and less intense for longer duration storms. The dimensionless mass rainfall curve for the modified SCS type II storm is presented in Figure 5. It is the same storm as the type II storm presented in Figure 4, only modified into a dimensionless form.

Table IV presents examples of the modified SCS type II storm. Unlike the balanced rainfall pattern, the modified type II storm has the same severity ratio for all duration storms. Also, the intensity is different for shorter storms than for longer duration storms. This trend is in agreement with the author's belief that there is a greater chance for high intensity rates in shorter duration storms than for longer duration storms.

The duration of storm that causes the maximum peak runoff rate is dependent on the shape of the unit hydrograph. Smaller basins with shorter lag times will have the maximum runoff rate produced by short duration storms. Larger watershed basins with longer lag times will have their maximum runoff rate produced by the longer duration storms. This seems to be a characteristic that is common in nature. None of the watershed basins tested have had the maximum peak runoff rate produced by the six-hour storm.

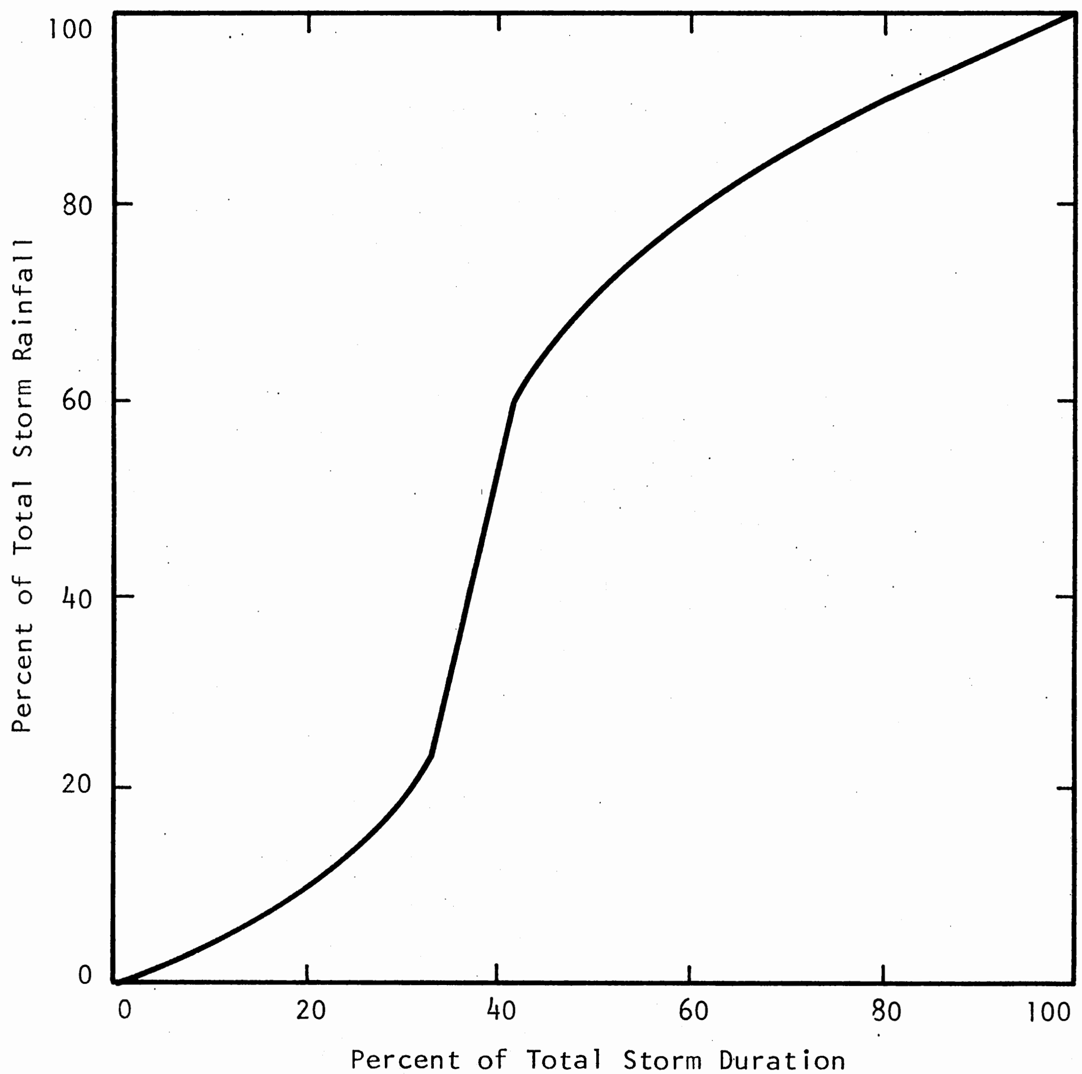

Figure 5. Dimensionless SCS Type II Storm

TABLE IV

DIMENSIONLESS SCS TYPE II STORM

Storm Duration	Total Rainfall, in.	Peak Intensity Rate	Duration of Peak Intensity	Severity Ratio
1 hr	3.79	15.58 in./hr	1.40" in 5.4 min	4.11
2 hr	4.86	9.99 in./hr	1.80" in 10.8 min	4.11
3 hr	5.40	7.40 in./hr	2.00" in 16.2 min	4.11
4 hr	5.74	5.90 in./hr	2.12" in 21.6 min	4.11
5 hr	6.06	4.98 in./hr	2.24" in 27.0 min	4.11
6 hr	6.45	4.42 in./hr	2.39" in 32.4 min	4.11
any	varies	varies	varies	4.11

$$\text{Severity Ratio} = \frac{\text{\% of total rainfall for most intense portion}}{\text{\% of total storm duration of most intense portion}}$$

3.4 The Abstraction Procedure

The third segment of a runoff model is the abstraction procedure to be used to account for rainfall losses. Several different abstraction procedures were reviewed with a modified SCS abstraction procedure selected.

The hydrologic cycle is presented in many textbooks concerning the rainfall and runoff process. These textbooks explain the initial losses and infiltration losses that are subtracted from the rainfall. Runoff is then produced from the excess rainfall after these abstractions are met. Such a graphical presentation will not be presented here, yet its concept is important to the understanding of the rainfall losses.

3.4.1 The City of Tulsa Abstraction Procedure

The City of Tulsa's design criteria manual (10) states that the abstraction procedure to be used in runoff models will be an initial loss of 0.5 inches and a 0.08 inch per hour constant abstraction loss thereafter. This abstraction procedure has been used extensively for the Snyder unit hydrograph used in the Corps of Engineers HEC-I runoff model. The procedure was not investigated for use in the TARM because it ignores differences in soil permeability. The Tulsa area has several types of soils, i.e., silts, clays, and sandy soils, and some other type of abstraction procedure was needed to model the losses caused by different soils.

3.4.2 The SCS Abstraction Procedure

The SCS used an empirical equation developed over many years of rural runoff modeling. This empirical equation is presented in Equation (7), which was developed from determining the runoff produced from 24-hour duration rainfalls:

$$Q = \frac{(P - 0.2\ S)^2}{(P + 0.8\ S)} \tag{7}$$

where

Q = total mass runoff, in inches;

P = total mass rainfall, in inches; and

S = potential abstraction, in inches.

Equation (8) is a determination of the potential abstraction that a watershed would expect to develop if enough rainfall were provided to completely saturate the soil so that total runoff could be expected. If the total rainfall in Equation (7) is great enough, the difference in total rainfall (P) and the total runoff (Q) would approach the potential abstraction (S).

$$S = \frac{1000}{CN} - 10 \tag{8}$$

where CN is the SCS curve number.

Figure 6 shows the application of the SCS abstraction losses as it relates to total rainfall (P). The initial abstraction loss (I_a) is 20 percent of the total abstraction (S), and the infiltration losses (F) approach 80 percent of the potential abstraction (S) as rainfall (P) accumulates.

$$I_a = 0.2 \times S \tag{9}$$

The abstraction procedure shown by Equation (7) and Figure 6 was used in the early model of the TARM. It also is used in the SCS TR-20 runoff model for watershed basins over 2000 acres. Its applications to a hydrograph procedure is detailed in the SCS handbook (8).

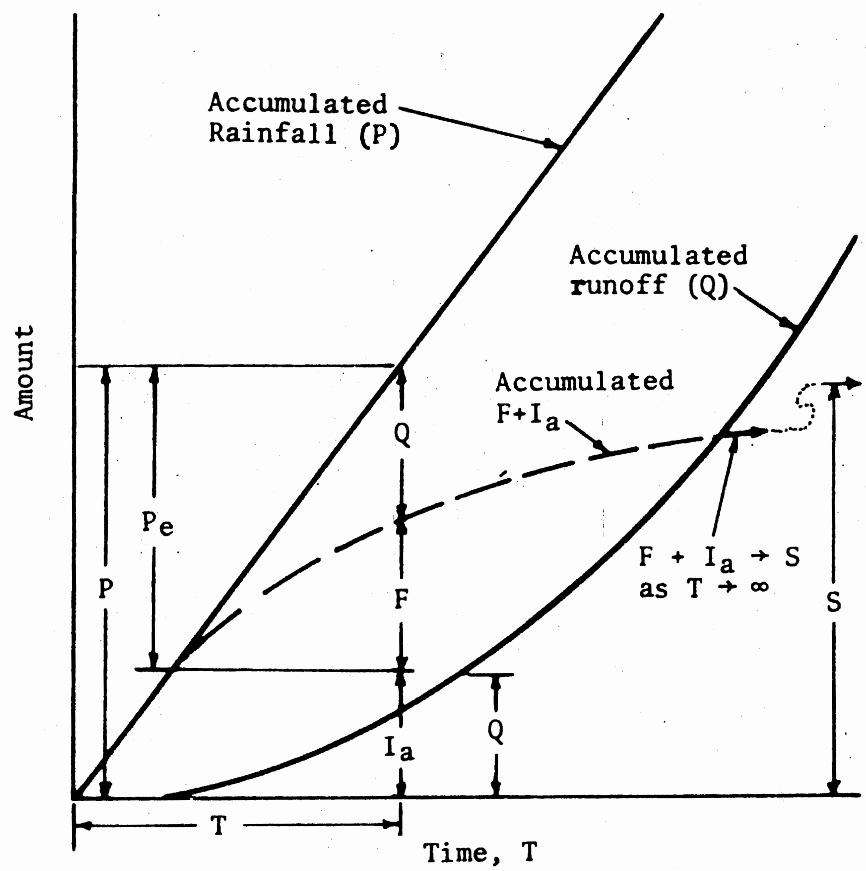

Figure 6. SCS Abstraction Procedure

Problems developed with the SCS abstraction procedure when it was used for small watershed and short duration storms. The total mass runoff (Q) is related only to total mass rainfall (P) and the potential abstraction (S). The abstraction procedure is not related to time in any way and thus does not work well with short high intensity storms.

Table V is an application of using the SCS abstraction procedure on a 100-year 2-hour Tulsa Area type II storm. Figure 7 is a graphical representation of the rainfall, abstraction losses, and runoff for the same storm. The SCS abstraction procedure causes a severe loss during the intense portion of the rainfall. The effect is to severely diminish the intense portion of the design storm. This causes a lower peak runoff rate to occur than would be expected.

3.4.3 Horton's and Holtan's Abstraction Procedure

For short duration storms, an abstraction procedure is needed that is related to time. By using the time parameter, it is hoped that the severe abstraction loss which occurred at the peak rainfall rate with the SCS abstraction procedure would not occur. Two methods were investigated using Horton's equation and Holtan's equation.

Horton (12) proposed in 1939 an exponential decay equation for an infiltration procedure. The equation is totally a function of time and is presented in Equation (10):

$$f = f_c + (f_o - f_c)e^{-kt} \tag{10}$$

where

TABLE V

SCS ABSTRACTION ON 100-YEAR, 2-HOUR STORM

Time (min)	SCS Mass Rainfall	SCS Mass Runoff	Delta Rainfall	Delta Abstraction	Delta Runoff
5	0.09	0	0.09	0.09	0
10	0.17	0	0.08	0.08	0
15	0.27	0	0.10	0.10	0
20	0.37	0	0.10	0.10	0
25	0.50	0	0.13	0.13	0
30	0.66	0.01	0.16	0.15	0.01
35	0.87	0.05	0.21	0.17	0.04
40	1.12	0.12	0.25	0.18	0.07
45	2.01	0.57	0.89	0.44	0.45
50	2.91	1.18	0.90	0.29	0.61
55	3.17	1.38	0.26	0.06	0.20
60	3.39	1.55	0.22	0.05	0.17
65	3.59	1.71	0.20	0.04	0.16
70	3.77	1.85	0.18	0.04	0.14
75	3.92	1.98	0.15	0.02	0.13
80	4.06	2.09	0.14	0.03	0.11
85	4.18	2.19	0.12	0.02	0.10
90	4.29	2.28	0.11	0.02	0.09
95	4.39	2.37	0.10	0.01	0.09
100	4.48	2.44	0.09	0.02	0.07
105	4.57	2.52	0.09	0.01	0.08
110	4.66	2.60	0.09	0.01	0.08
115	4.75	2.68	0.09	0.01	0.08
120	4.85	2.76	0.10	0.02	0.08

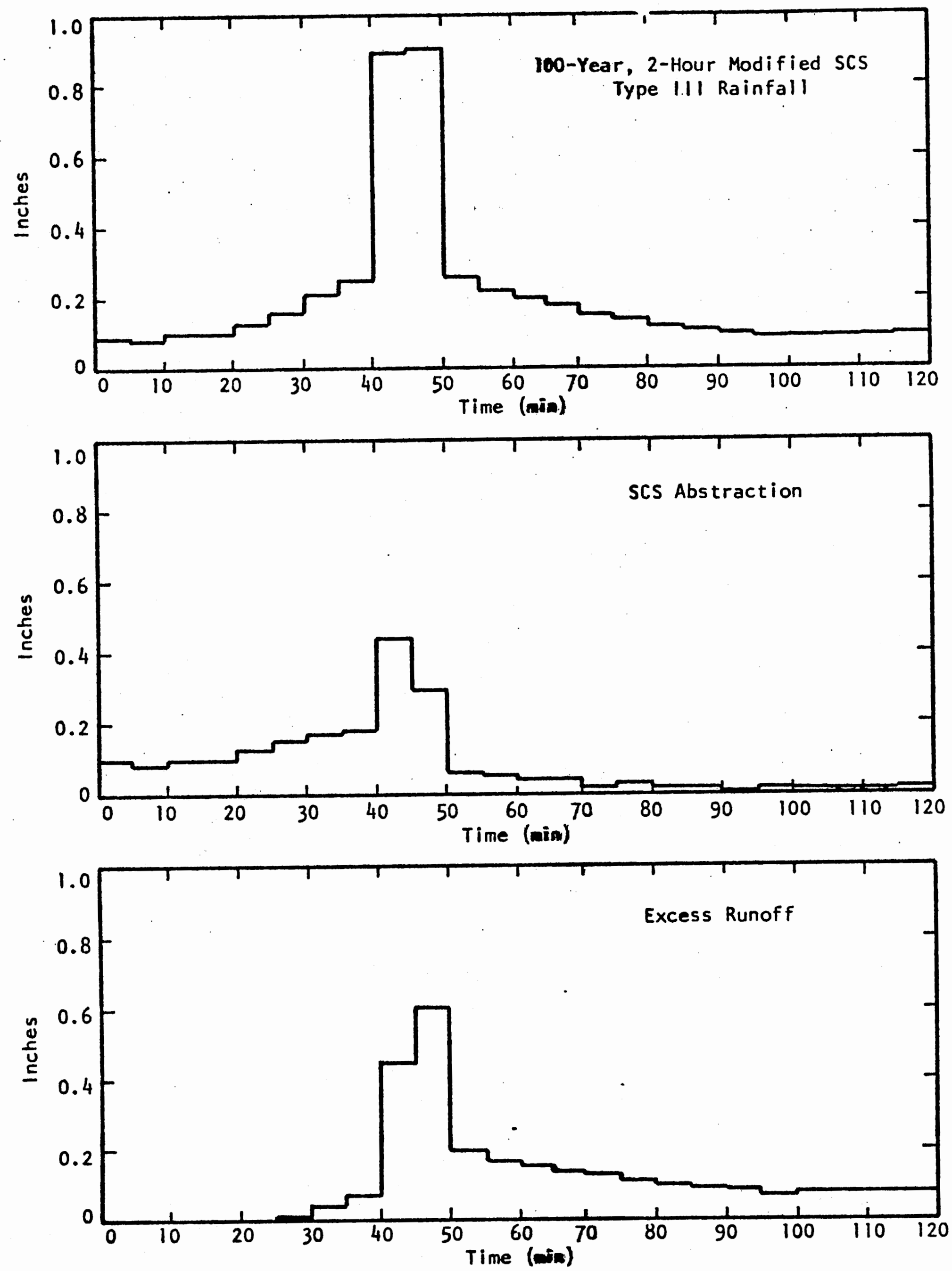

Figure 7. SCS Abstraction

f = infiltration rate, inches per hour;

f_c = final infiltration rate, inches per hour;

f_o = initial infiltration rate, inches per hour;

t = time, seconds or hours;

k = decay coefficient, seconds^{-1} or hours^{-1}; and

e = natural logarithm base.

The major advantage of Horton's equation is its simplicity. It is a function of time only and can easily be used in a model. Horton's equation is used in the Storm Water Management Model (SWMM), as well as some other models.

Horton's equation was not used in the TARM because the user has to estimate f_c, f_o, and k, which is not possible to do accurately without rainfall and runoff data. Tulsa does have rainfall data but does not have sufficient runoff data to determine the required parameters.

Another problem associated with using Horton's equation is that there is no universal table referencing values for the variables to soil types, slopes, or other parameters that are used in runoff models. So applying the equation to the SCS input parameters was not possible.

Holtan (13) proposed in 1961 another empirical infiltration equation. Holtan recognized that as the soil pores fill, the infiltration rate decreases until it reaches a final soil percolation rate. The equation Holtan proposed related the infiltration rate to the unsaturated pore volume remaining in the soil. Holtan's equation is:

$$f = AF_p(t)^c + f_c \tag{11}$$

where

f = instantaneous infiltration rate;

$F_p(t)$ = pore volume remaining at time t;

f_c = final infiltration rate;

c = experimental exponent; and

a = experimental coefficient.

Holtan's equation had several problems associated with it. It was illustrated by Holtan that the coefficient "a" can vary significantly with the antcedent soil moisture. This makes it difficult to estimate. Holtan's equation is also difficult to use in a simple model, because the relationship between infiltration and remaining pore water volume must be known. The method was dropped due to its difficulty.

3.4.4 The Modified SCS Abstraction Procedure

The abstraction procedure that was selected to be used in the TARM was again some modified form of the SCS procedure. The initial abstraction (I_a) was set at 20 percent of the potential abstraction (S). The infiltration losses, 80 percent of the potential abstraction (S), were then abstracted in a constant decaying rate over a 12-hour period of time.

This modified form of the SCS abstraction procedure relates the losses to time and thus does not develop the problem of severe losses during the peak rainfall rates that occur with Equation (7). The modified SCS abstraction procedure is shown in Figure 8. It is also presented in Equation (12):

$$Q = P - S + \frac{(0.8S)((t_o + 720) - t)^2}{518400} \tag{12}$$

where

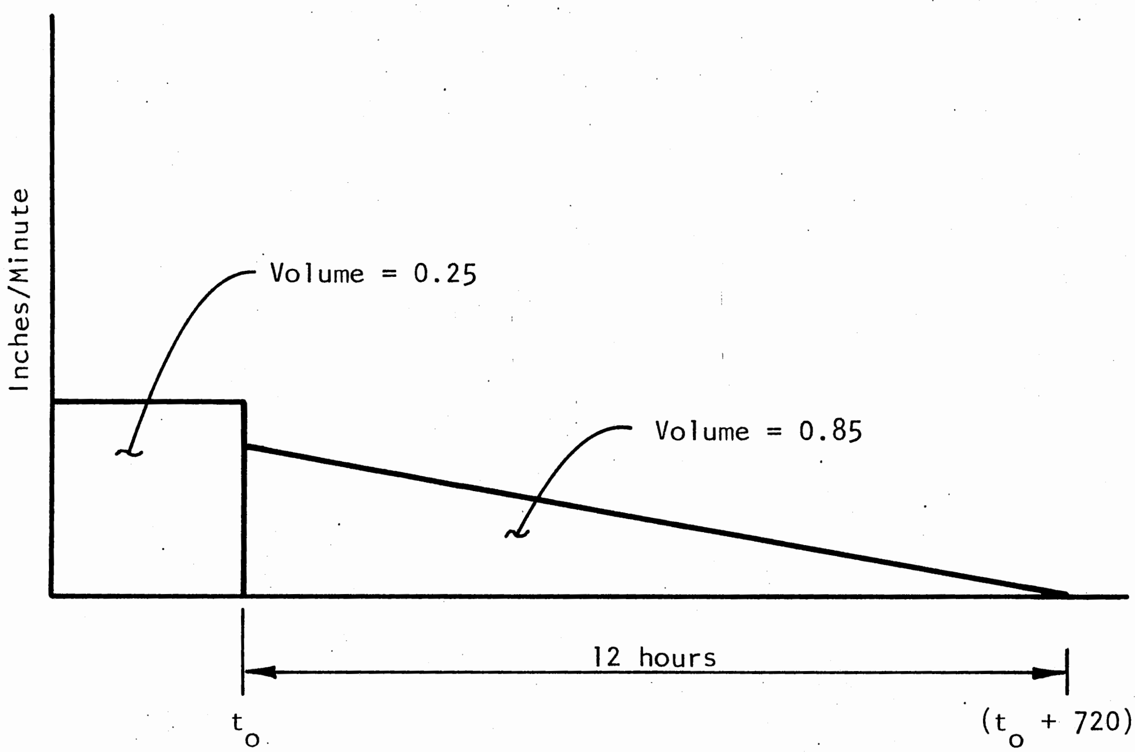

Figure 8. Modified SCS Abstraction

Q = total mass runoff, in inches (must be greater than zero), at time t;

P = total mass rainfall, in inches at time t;

S = potential abstraction, in inches; and

t_o = time at which the initial abstraction (I_a) is met.

Table VI shows the application of the modified SCS abstraction procedure on the 100-year two-hour SCS type II storm. Figure 9 shows graphically how the method does not diminish the intense portion of the design storm that was shown in Figure 7. This modified SCS abstraction procedure models more accurately the abstraction losses for short duration storms, yet maintains some of the work the SCS developed in their runoff model.

3.5 The Complete Model

The TARM consists of three segments. The SCS dimensionless curvilinear unit hydrograph or the triangular unit hydrograph that was developed by Victor Mockus. The unit hydrograph is determined by either the time of concentration (T_c) of the watershed or by the synthetic lag equation.

The design rainfall pattern is a modified SCS type II six-hour storm. The type II storm is modified to a dimensionless form so that it can be used for short duration storms.

The abstraction procedure is a modified SCD abstraction rate. The initial loss is based on the SCS abstraction loss. The abstraction procedure uses the same input parameter as the SCS procedure. The modified SCS abstraction is related to time rather than rainfall, as first developed by the SCS. The modified procedure was developed for TARM.

TABLE VI

MODIFIED SCS ABSTRACTION ON 100-YEAR, 2-HOUR STORM

Time (min)	SCS Mass Rainfall	SCS Mass Runoff	Delta Rainfall	Delta Abstraction	Delta Runoff
5	0.09	0	0.09	0.09	0
10	0.17	0	0.08	0.08	0
15	0.27	0	0.10	0.10	0
20	0.37	0	0.10	0.10	0
25	0.50	0	0.13	0.13	0
30	0.66	0.13	0.16	0.03	0.13
35	0.87	0.31	0.21	0.03	0.18
40	1.12	0.54	0.25	0.02	0.23
45	2.01	1.40	0.89	0.03	0.86
50	2.91	2.27	0.90	0.03	0.87
55	3.17	2.51	0.26	0.02	0.24
60	3.39	2.70	0.22	0.03	0.19
65	3.59	2.87	0.20	0.03	0.17
70	3.77	3.03	0.18	0.02	0.16
75	3.92	3.15	0.15	0.03	0.12
80	4.06	3.27	0.14	0.02	0.12
85	4.18	3.36	0.12	0.03	0.09
90	4.29	3.45	0.11	0.02	0.09
95	4.39	3.52	0.10	0.03	0.07
100	4.48	3.59	0.09	0.02	0.07
105	4.57	3.65	0.09	0.03	0.06
110	4.66	3.72	0.09	0.02	0.07
115	4.75	3.78	0.09	0.03	0.06
120	4.85	3.86	0.10	0.02	0.08

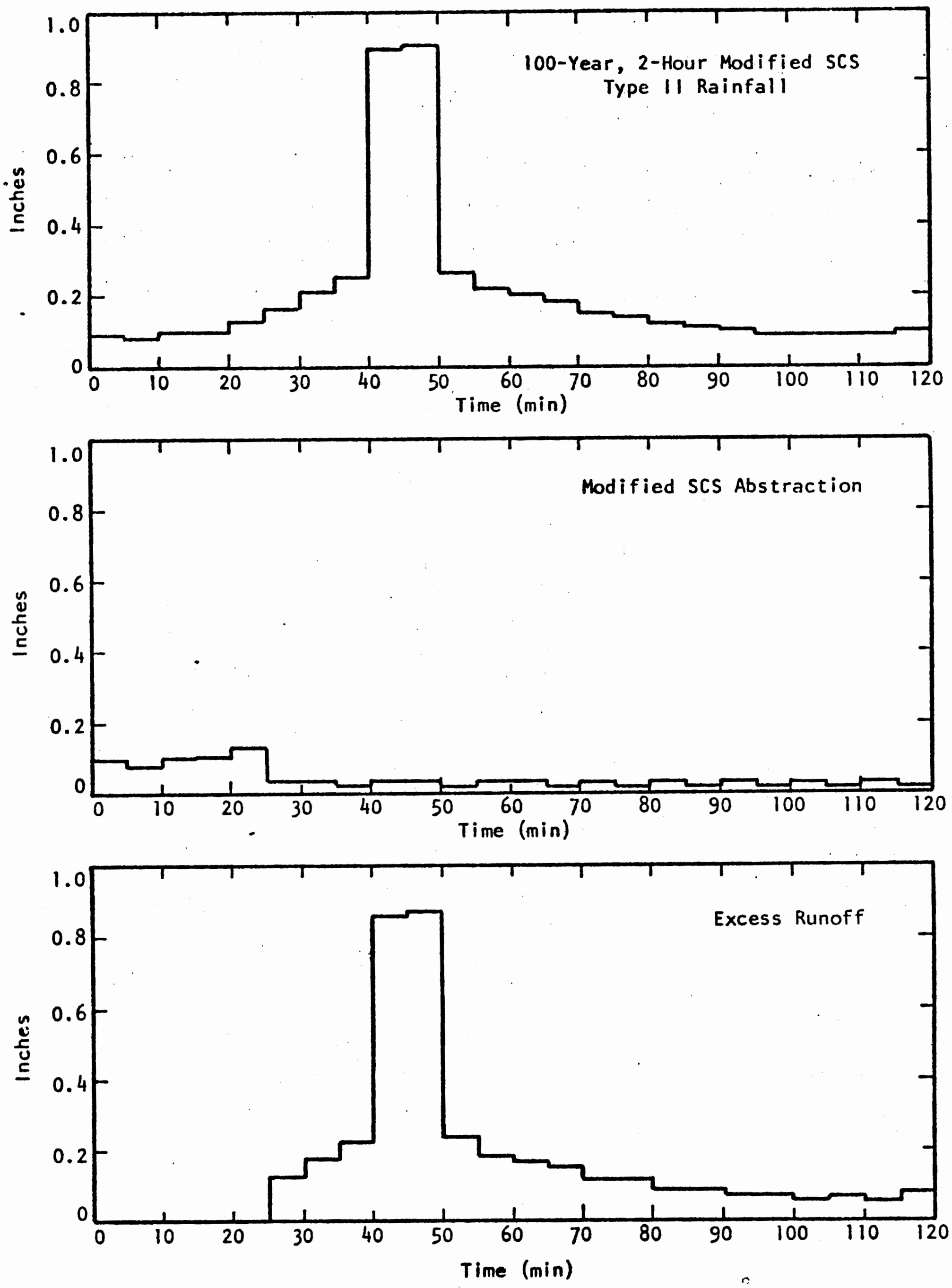

Figure 9. Modified SCS Abstraction

CHAPTER IV

APPLICATION OF THE TULSA AREA RUNOFF MODEL

4.1 General

The TARM was developed to be used as a hydrograph method of determining peak runoff rates and volumes of runoff for small watersheds. Since the model uses the SCS curve number approach, it can easily be used for urbanized watersheds. The runoff model was developed for watersheds of less than 2000 acres where the SCS TR-20 model is not applicable.

The TARM is presently written for two computer systems. One program is written in FORTRAN language for use on a Honeywell computer. The other is written for a Texas Instrument model 59 desktop calculator. Both programs are written in segments so that they can easily be adaptive to other computer systems.

The development of a subdivision would be a good example for the use of the TARM. The model could be used to determine the runoff characteristics of the watershed in its existing condition. The existing condition may or may not be in a state of some urbanization. The model can then be used for determining the runoff characteristics in the proposed developed state. The change in runoff characteristics will aid the engineer in determining the impact the development will have on the watershed.

The usual impact development has on a watershed is that it increases the peak runoff rate. The total volume of runoff will increase and the travel time of the runoff through the watershed will be less. Decreasing the travel time, or lag time, through the watershed is the single largest impact by development.

Figure 10 shows what happens to a hydrograph when a watershed changes from a natural to an urbanized state. The time to peak for the urbanized hydrograph is shorter than what it was for the natural hydrograph. The volume under the urbanized hydrograph and above the natural hydrograph (shown in cross hatch) is the runoff that is occurring sooner than it did in its natural condition.

If this volume between the two hydrographs were detained in a small reservoir and then released in such a manner that the release rate followed the natural runoff rate, then the reservoir would attenuate the peak back to the natural condition. There still would be an increase in the total volume of runoff on the watershed, but the peak runoff rate and the time that the peak occurred would simulate the natural condition.

The TARM can be used to design such a reservoir to offset the impact of urbanization. The reservoir is called a detention pond because it detains the peak of the urbanized runoff.

4.2 Detention Pond Design

Three pieces of information are needed to design a detention pond. They are:

1. The release rate for a given frequency of flood, which is the natural runoff rate from the watershed.

2. The urbanized runoff rate from the watershed for the same frequency flood.

3. The volume of storage required to offset the increase in runoff.

In order to determine the volume of storage required to offset urbanization, a runoff hydrograph for both the natural and developed condition is needed. The volume of runoff between the natural and urbanized

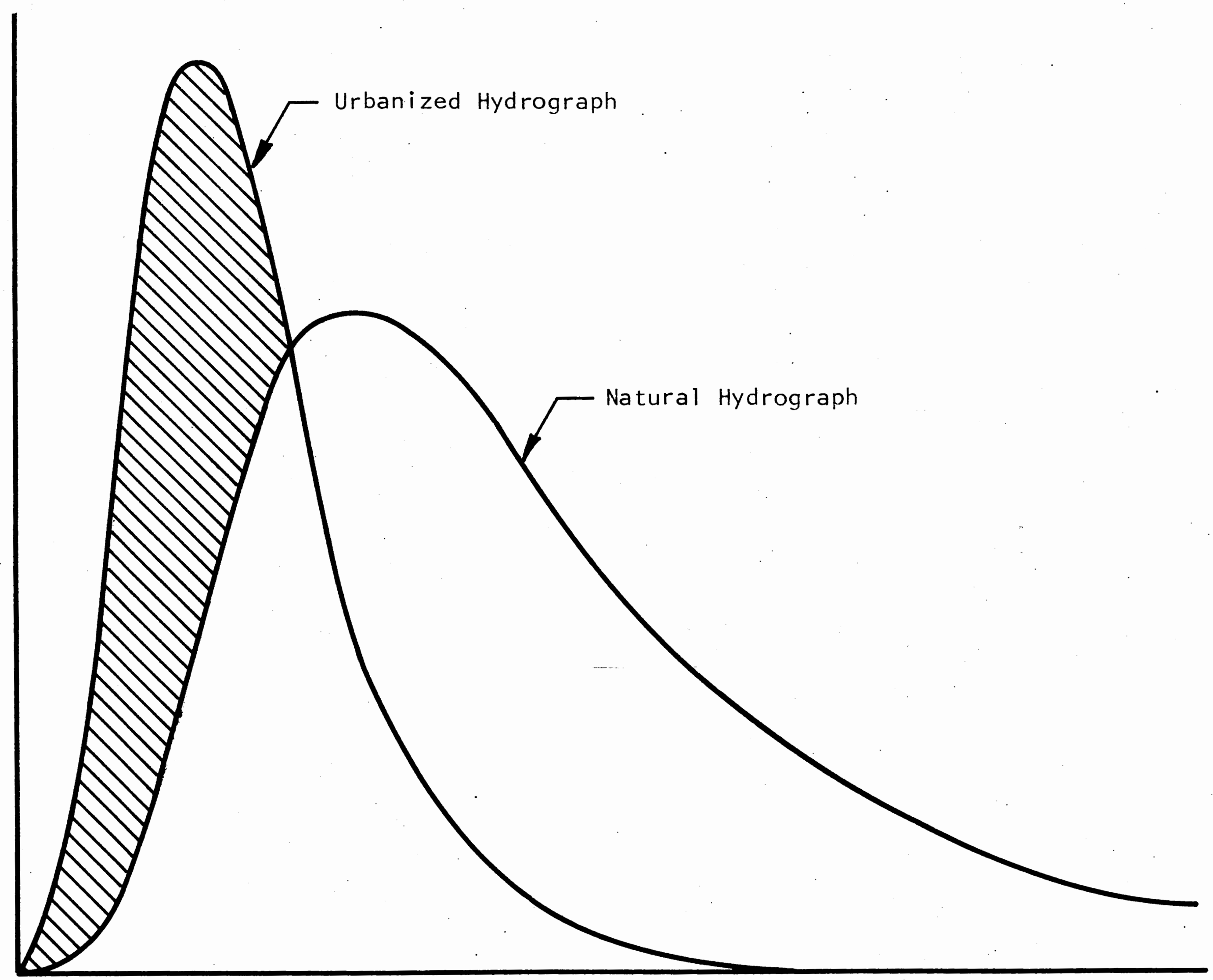

Figure 10. Natural and Urbanized Hydrograph

hydrograph will be an approximate volume required for storage. Usually the actual volume will be slightly greater because the release rate will not be a replica of the natural release rate.

Since different duration storms can be expected to cause different peak runoff rates, several storms of different duration but with the same frequency will need to be evaluated to determine the critical storm. The 1-, 2-, 3-, 4-, 5-, and 6-hour storms are applied in this example to see which will cause the greatest peak runoff.

Figure 11 is a graphical plot of the peak runoff rates for a watershed with a 100-year frequency family of storms. The family of storms is the 1- through 6-hour storms. The watershed is a 351-acre basin and will be referred to as basin A. The bottom figure is the volume difference between the natural and the urbanized hydrographs for each duration of storm.

Basin A, in its natural condition, had a maximum peak runoff rate occurring at the 4-hour storm. In the urbanized case, the maximum peak runoff rate occurred during the 2-hour storm. Yet the maximum difference between the two hydrographs occurred during the 3-hour storm.

Since any of the storms have a likely chance of occurring, the one that produces the maximum difference in runoff will be the controlling hydrograph. For basin A, the 3-hour storm had the greatest volume difference of 80.9 acre feet of runoff. The urbanized peak runoff rate for the 3-hour storm for basin A was 932 cfs and the natural peak runoff rate was 489 cfs.

The next step in the design of a detention pond would be to determine the natural release rate. It would be impossible to determine the natural release rate for every frequency of storm with every possible duration of storm. Every storm in nature is different and will interact differently.

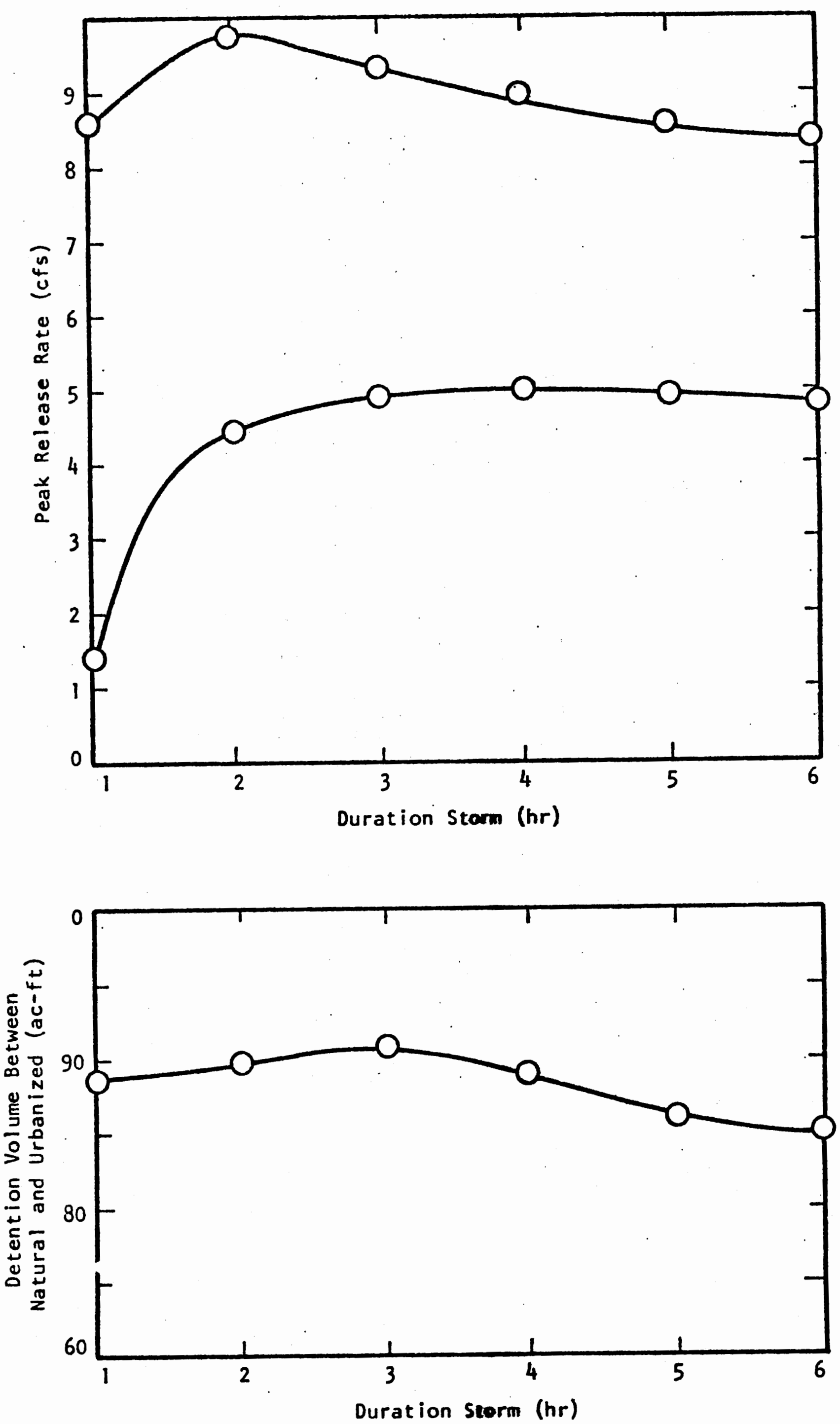

Figure 11. Peak Runoff Rates for Basin A

What can be acceptable is some type of average release rate that can offset the impact of urbanization during most storms.

To determine the release rate, an average release rate of the one- through six-hour storms is selected. For basin A, this would be 423 cfs for a 100-year frequency family of storms. This gives a higher release rate than the one-hour natural release rate, but would be lower for the two-, three-, four-, five-, and six-hour storms natural release rate.

A new volume of storage must be estimated because the 80.9 acre feet of storage was based on a release rate of 489 cfs. A simple ratio is then used to correct the estimated volume. The ratio of the natural release rate to the average release rate times the volume is used. For basin A, this would be (489/423) x 80.9 = 93.5 acre feet.

This form of modifying the volume of storage is possible because the time between the peak of the natural hydrograph and the peak of the urbanized hydrograph is close to being the same regardless of the duration of the storm used. This time difference is the change in the basin lag between the natural and urbanized state and is independent of the duration of the storm. By making the assumption that each point on the natural hydrograph is reduced by the ratio of the change in peak, an increase in volume can be estimated.

The detention basin should then release a peak of no more than 423 cfs when an urbanized hydrograph with a peak of 975 cfs is routed through the detention pond. The required storage will be close to the 93.5 acre feet of storage estimated, but will vary depending on the actual release rate.

If the requirement of the location were to offset the impact of urbanization for every frequency of storm, several families of hydrographs would have to be made, one for each frequency of storm for which protection must be provided. A detention pond should be sized at the low and high ends of the frequency curve. The City of Tulsa presently requires a hydrograph determination for the 5- and 100-year storms for sizing the detention pond. A 500-year storm is also routed through the detention pond to size the overflow spillway to prevent breaching of the earth embankment.

4.3 Actual Basins

Three actual proposed development sites were modeled using the TARM. The basins ranged in size from 351 acres to the smallest basin of 7.6 acres. An on-site detention pond was sized for each basin based on the procedure described. The three basins were then compared to the graphical procedure presented in TR-55 that estimates peak runoff rates for a small watershed basin.

The SCS TR-55 presents a method (see Appendix) for determining the 24-hour peak runoff rate for small watersheds of less than 2000 acres. The input parameters for the graphical method are very much the same as those used for the TARM. The method is too detailed to present here, but consists of a series of modifications to a peak runoff factor based on the runoff curve number (CN) and the basin size. The procedure for modifying the runoff peak rate is based on several factors:

1. The size of the watershed to a computed size based on the watershed's hydraulic length.

2. The average slope of the watershed.

3. The percentage of the hydraulic length modified from its natural state.

4. The percentage of impervious area.

The volume of storage computed by the SCS TR-55 graphical method is strictly based on the ratio of the inflor to outflow peak runoff rates, the size of the basin, and the number of inches of runoff. There is no provision for the change in lag time of the basin. The storage method is an approximate procedure for single stage structures.

4.3.1 Basin A

Basin A is the same 351 acre watershed that was described earlier in this chapter. Its analysis is presented in Table VII. Basin A is located in a sandy to a silty-sandy type of soil. Its slope is moderate at 4 percent. The storage required for the 100-year storm is 94 acre-feet.

4.3.2 Basin B

Basin B, whose analysis is presented in Table VIII, is a 90-acre watershed with a fairly heavy impact due to urbanization. There is a very large percentage of natural channel eliminated (90%), and impervious area (40%). Basin B has a required storage for the 100-year storm of 14.9 acre-feet.

4.3.3 Basin C

Basin C is a very small development of only 7.6 acres. Its analysis is presented in Table IX. It has a mildly steep slope of 5 percent with moderate urbanization. Due to its extremely small time increment, the hydrograph runs exceeded the capacity of the computer's available storage for the four-hour storm. Basin C had a storage requirement for the 100-year storm of 0.44 acre-feet.

TABLE VII

RUNOFF FOR BASIN A

Area	351 acres
Length of Watershed	7660 feet
Upland Land Slope	1 percent

	Natural Condition	Urbanized Condition
Curve Number (CN)	73	83
% Natural Channel Eliminated	0	41
% Impervious Area	0	51

Rainfall Duration	5-Year	100-Year
1 hr	2.28	3.78
2 hr	2.81	4.86
3 hr	3.15	5.38
4 hr	3.34	5.74
5 hr	3.53	6.06
6 hr	3.71	6.40

Runoff

	5-Year			100-Year		
Storm Duration	Nat. (cfs)	Urb. (cfs)	Storage (ac-ft)	Nat. (cfs)	Urb. (cfs)	Storage (ac-ft)
1 hr	76	510	46.4	137	861	78.6
2 hr	227	549	45.5	446	975	79.8
3 hr	223	529	48.4	489	932	80.9
4 hr	277	503	43.5	496	895	79.0
5 hr	272	482	41.8	488	860	76.3
6 hr	266	466	40.1	484	841	75.1

TABLE VIII

RUNOFF FOR BASIN B

Area	90 acres
Length of Watershed	2800 ft
Upland Land Slope	4 percent

	Natural Condition	Urbanized Condition
Curve Number (CN)	75	85
% Natural Channel Eliminated	0	90
% Impervious Area	0	40

Rainfall Duration	5-Year	100-Year
1 hr	2.28	3.78
2 hr	2.81	4.86
3 hr	3.15	5.38
4 hr	3.34	5.74
5 hr	3.53	6.06
6 hr	3.71	6.40

Runoff

	5-Year			100-Year		
Storm Duration	Nat. (cfs)	Urb. (cfs)	Storage (ac-ft)	Nat. (cfs)	Urb. (cfs)	Storage (ac-ft)
1 hr	199	446	8.1	376	771	13.0
2 hr	159	395	8.2	372	715	13.5
3 hr	191	336	5.9	348	604	11.8
4 hr	177	280	5.7	326	510	10.9
5 hr	166	242	5.6	307	443	10.1
6 hr	159	214	5.4	297	396	9.6

TABLE IX

RUNOFF FOR BASIN C

Area	7.6 acres
Length of Watershed	520 feet
Upland Land Slope	5 percent

	Natural Condition	Urbanized Condition
Curve Number (CN)	79	84
% Natural Channel Eliminated	0	100
% Impervious Area	0	43

Rainfall Duration	5-Year	100-Year
1 hr	2.28	3.78
2 hr	2.81	4.86
3 hr	3.15	5.38
4 hr	3.34	5.74
5 hr	3.53	6.06
6 hr	3.71	6.40

Runoff

	5-Year			100-Year		
Storm Duration	Nat. (cfs)	Urb. (cfs)	Storage (ac-ft)	Nat. (cfs)	Urb. (cfs)	Storage (ac-ft)
1 hr	44	65	.284	81	113	.438
2 hr	38	41	.141	69	76	.262
3 hr	30	30	.135	55	56	.201
4 hr	*	*	*	*	*	*

*Overflowed the 16k memory capacity of the Honeywell computer.

4.4 Discussion of Comparisons

Table X presents the comparisons of each of the three watersheds with the TR-55 graphical method. Both the 5- and 100-year storms are presented along with the required storage to provide protection from urbanization for both methods.

The TARM gave reasonably close results to the graphical method for basins A and B. For basin C, the TARM gave a lower peak runoff rate for all cases and a much lower storage requirement. The large storage requirement produced by the graphical method is caused by the much larger peak runoff rates, both in the natural and in the urbanized conditions. The natural release rates based on the graphical method for basin C are slightly lower than the urbanized peak runoff rates for the TARM.

It must be pointed out that the graphical method is not a hydrograph method and should not be expected to provide the actual storage required. The graphical method will provide a peak runoff rate for a watershed and based on those peak runoff rates will approximate a storage.

TABLE X

COMPARISON OF MODIFIED SCS RUNOFF MODEL
WITH SCS TR-55 APPENDIX E

	TARM	Appen. E TR-55
Basin A		
5-Year Natural Release Rate (cfs)	224	172
5-Year Urbanized Peak Inflow (cfs)	549	488
Storage (acre-feet) 5-Year	48.2	38.5
100-Year Natural Release Rate (cfs)	423	401
100-Year Urbanized Peak Inflow (cfs)	975	1005
Storage (acre-feet) 100-Year	94	69.5
Basin B		
5-Year Natural Release Rate (cfs)	175	91
5-Year Urbanized Peak Inflow (cfs)	446	289
Storage (acre-feet) 5-Year	9.1	11.0
100-Year Natural Release Rate (cfs)	338	206
100-Year Urbanized Peak Inflow (cfs)	771	584
Storage (acre-feet) 100-Year	14.9	21.2
Basin C		
5-Year Natural Release Rate (cfs)	37	46
5-Year Urbanized Peak Inflow (cfs)	65	113
Storage (acre-feet) 5-Year	0.33	0.73
100-Year Natural Release Rate (cfs)	68	100
100-Year Urbanized Peak Inflow (cfs)	113	232
Storage (acre-feet) 100-Year	0.52	1.44

CHAPTER V

RESULTS AND CONCLUSIONS

5.1 The Model

Hydrograph procedures have become the acceptable procedures in modern hydrology for use in determining runoff rates for watersheds. They are used almost extensively for design of all hydraulic structures of any significant size.

The reason for the development of the model was a valid one. The Soil Conservation Service had made an effort to modify its extensive modeling of the rural watershed to account for the effects of urbanization. They developed a model to be used for watersheds of either an urbanized or rural state for basins greater than 2000 acres. The model they developed was the TR-20 runoff model. The basis for developing the TARM was to use the information that the Soil Conservation Service published in TR-55 and to modify it for use on small watersheds. This model is the result of that attempt.

The TARM has taken over two years to develop into its present state. Its development has progressed slowly as each parameter was carefully studied. It presently is being used on a limited scale by the engineering staff at the City of Tulsa as well as several consultants within the area. It has been used both in the determination of peak runoff rates as well as designing on-site detention ponds.

The method has been compared to Snyder's synthetic runoff model as well as the modified rational procedure. These comparisons have not always produced pleasing results, but have always aided the engineer in the design of the project.

There has been some argument as to the use of hydrograph theory on small watersheds. It is believed by some hydrologists to be an invalid appraoch and that the theory of the unit hydrograph breaks down on small basins. Every effort has been made to study the theoretical validity of each segment of the model.

5.2 The Use of the Model

The TARM has been used for the design of on-site detention ponds. The design of such retarding basins was one of the reasons this model was developed. But care should be taken in the use of such structures in every case. There is much validity to the argument regarding the effectiveness of on-site detention structures versus regional detention structures for floodplain management. Nonetheless, on-site detention structures do have a place in the control of flooding in an urbanized environment.

As with any computer model, care should be taken to prevent the user from becoming too dependent on the machine. The runoff model was developed to aid the designer in a rational and well considered approach in watershed runoff modeling. The model is simply an aid to its user.

CHAPTER VI

FUTURE STUDIES

During the investigation of the parameters of the runoff model, it became apparent that there was a scarcity of shelf-ready design storms that could be used in the application of short duration analysis. The bulk of the publications dealt with point rainfall and point-to-area rainfall frequency ratios (14).

The U.S. Department of Commerce has published its results of interduration precipitation relations (15), but its report dealt with long duration storms in the southeast region of the United States. Further studies need to be performed to investigate the interduration precipitation relations for short duration storms in the midwest region.

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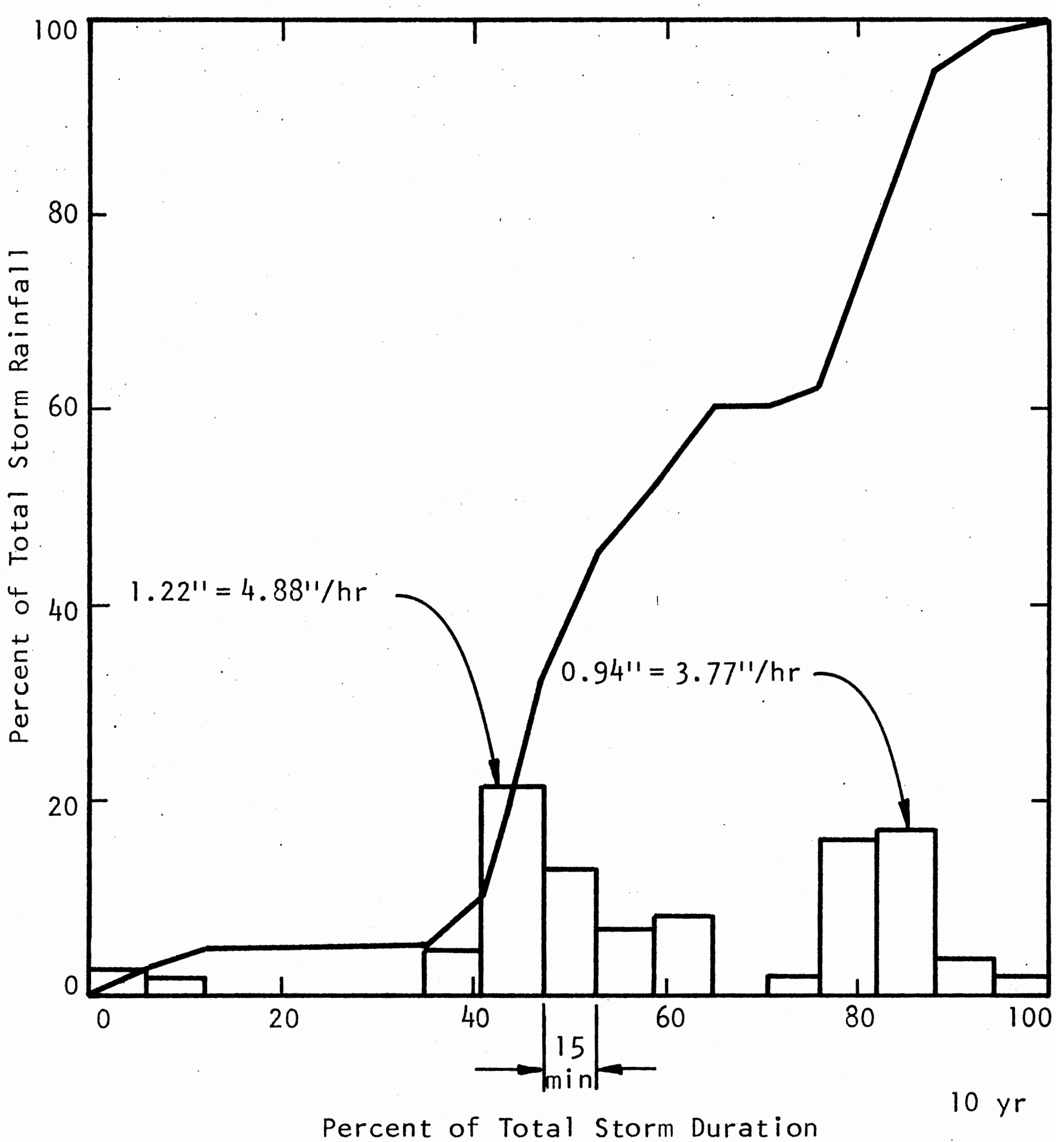
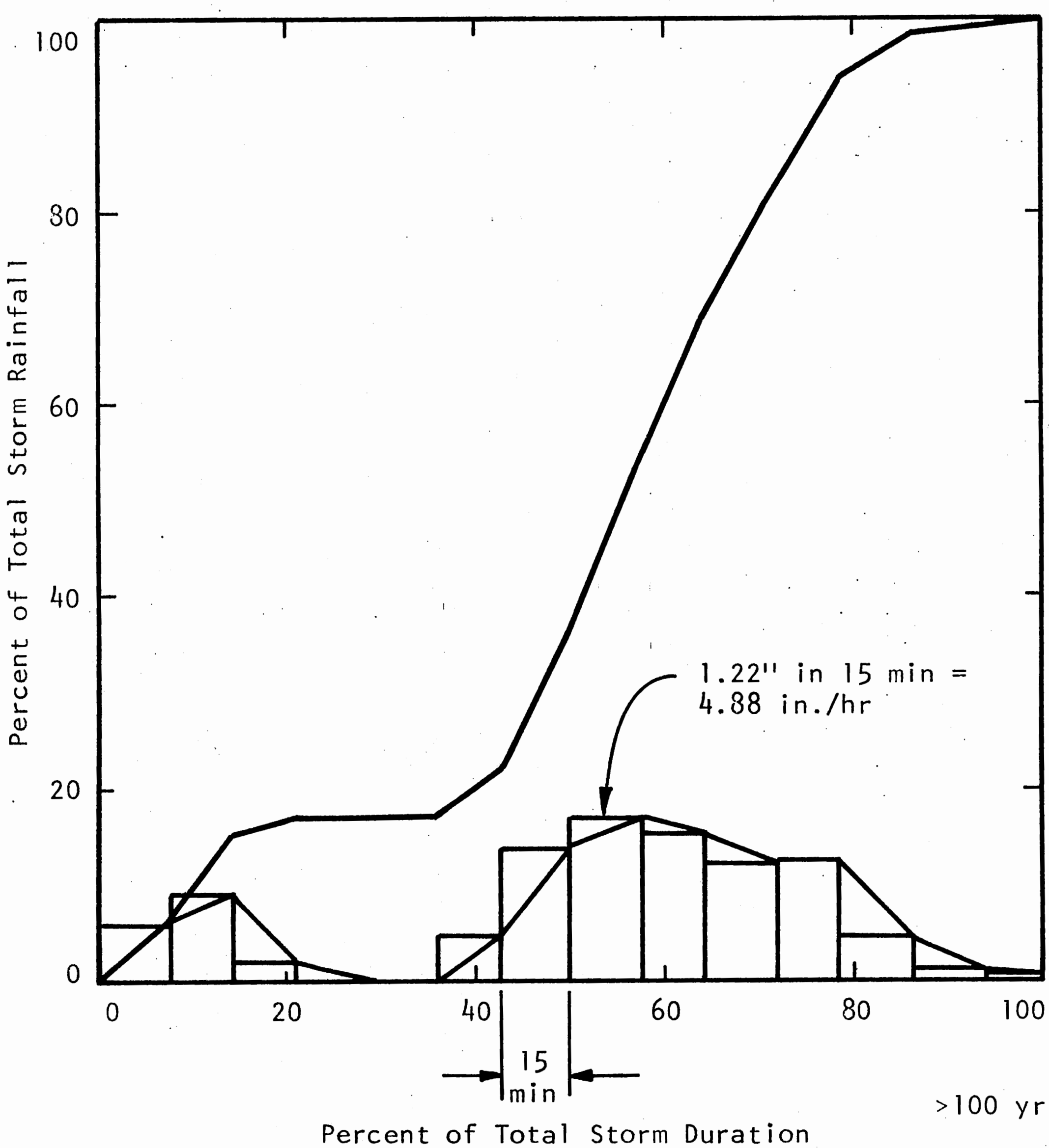
APPENDIX

RAINFALL PATTERNS



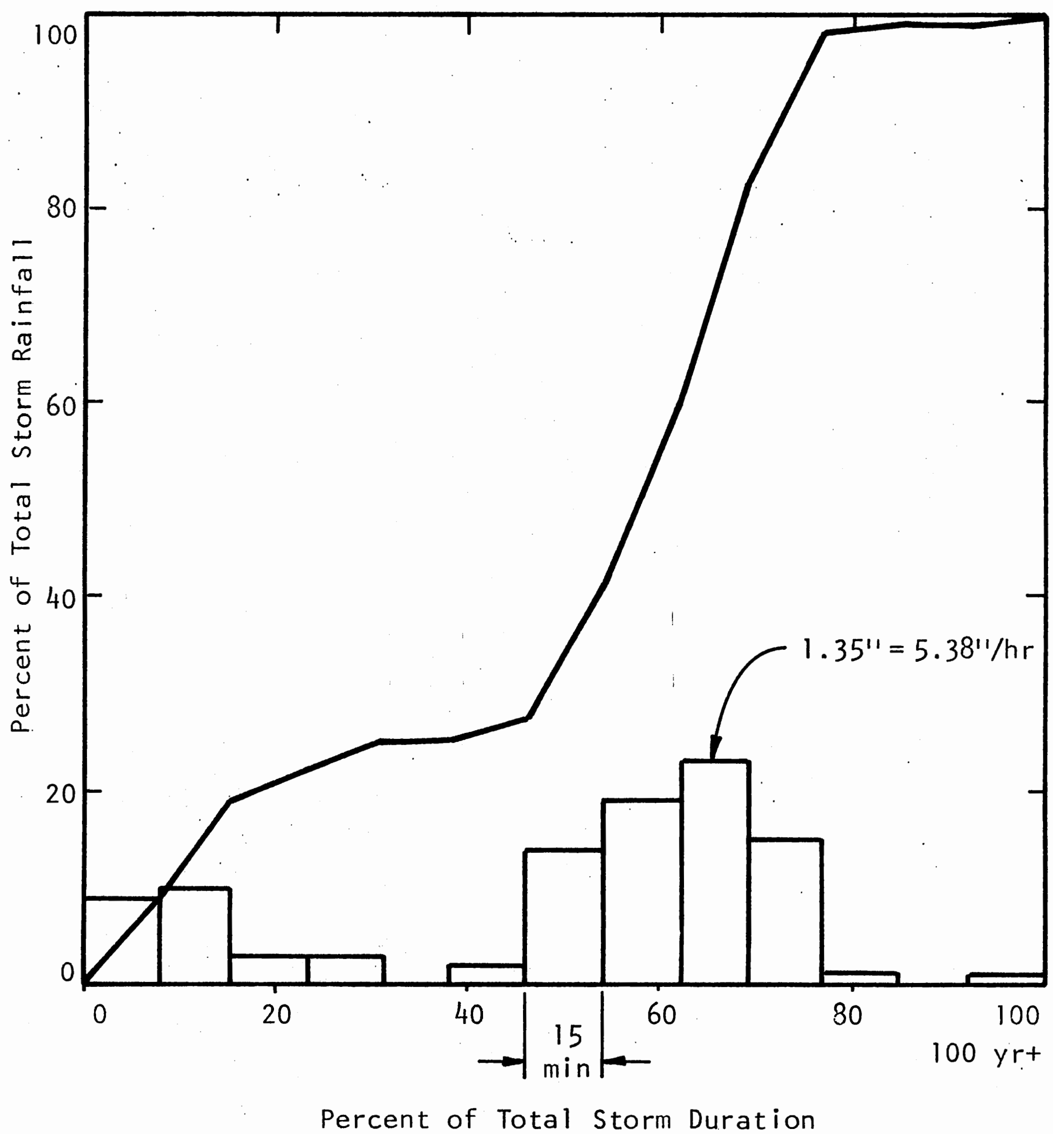
May 9, 1970
Gage No. 5
61st and Mingo
Duration 4.25 hrs
Total Rainfall 5.55 in.

Figure 12. Rainfall Pattern for May 9, 1970, Gage No. 5 (10-Year Frequency)



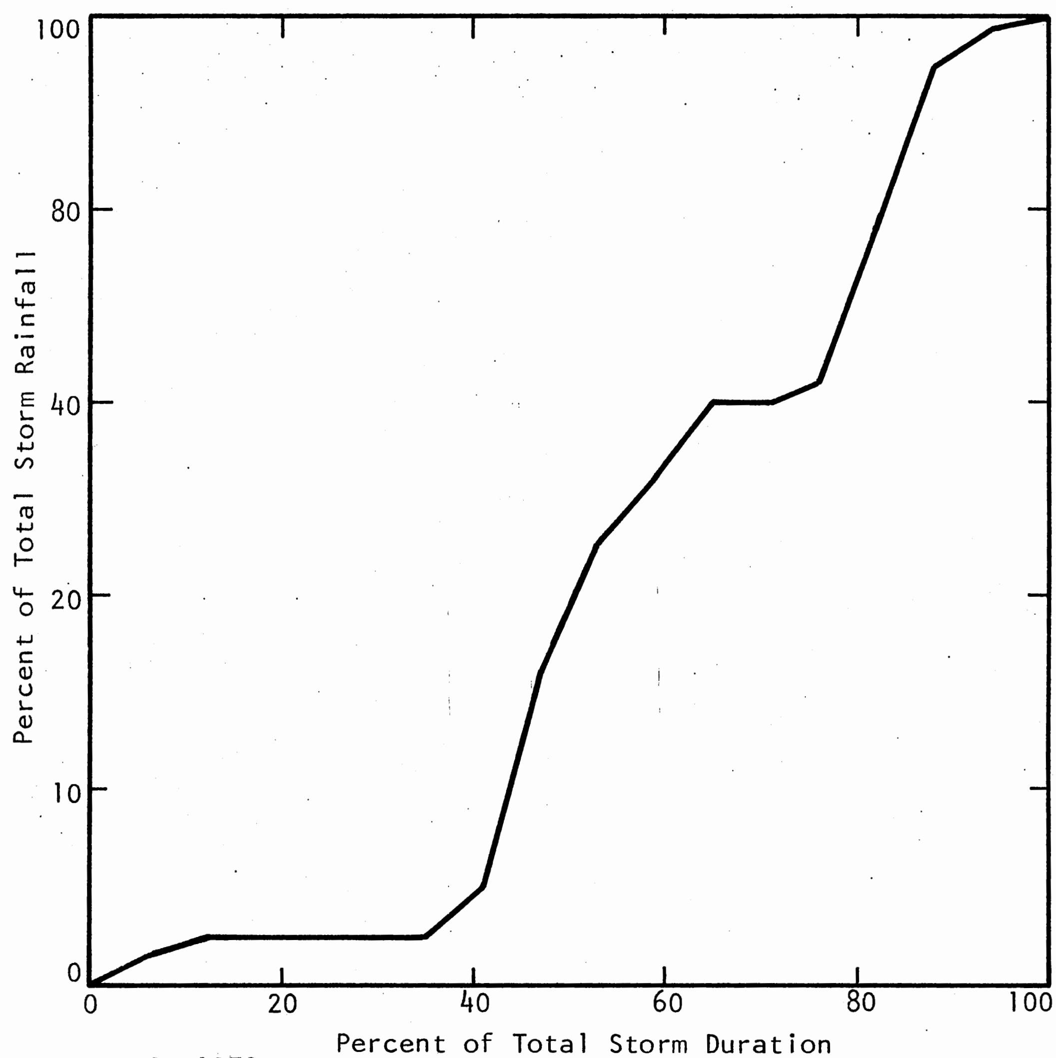
May 30, 1976
Gage No. 5
61st and Mingo
Duration Storm 3.5 hrs
Total Rainfall 7.15 in.

Figure 13. Rainfall Pattern for May 30, 1976, Gage No. 5
(>100-Year Frequency)



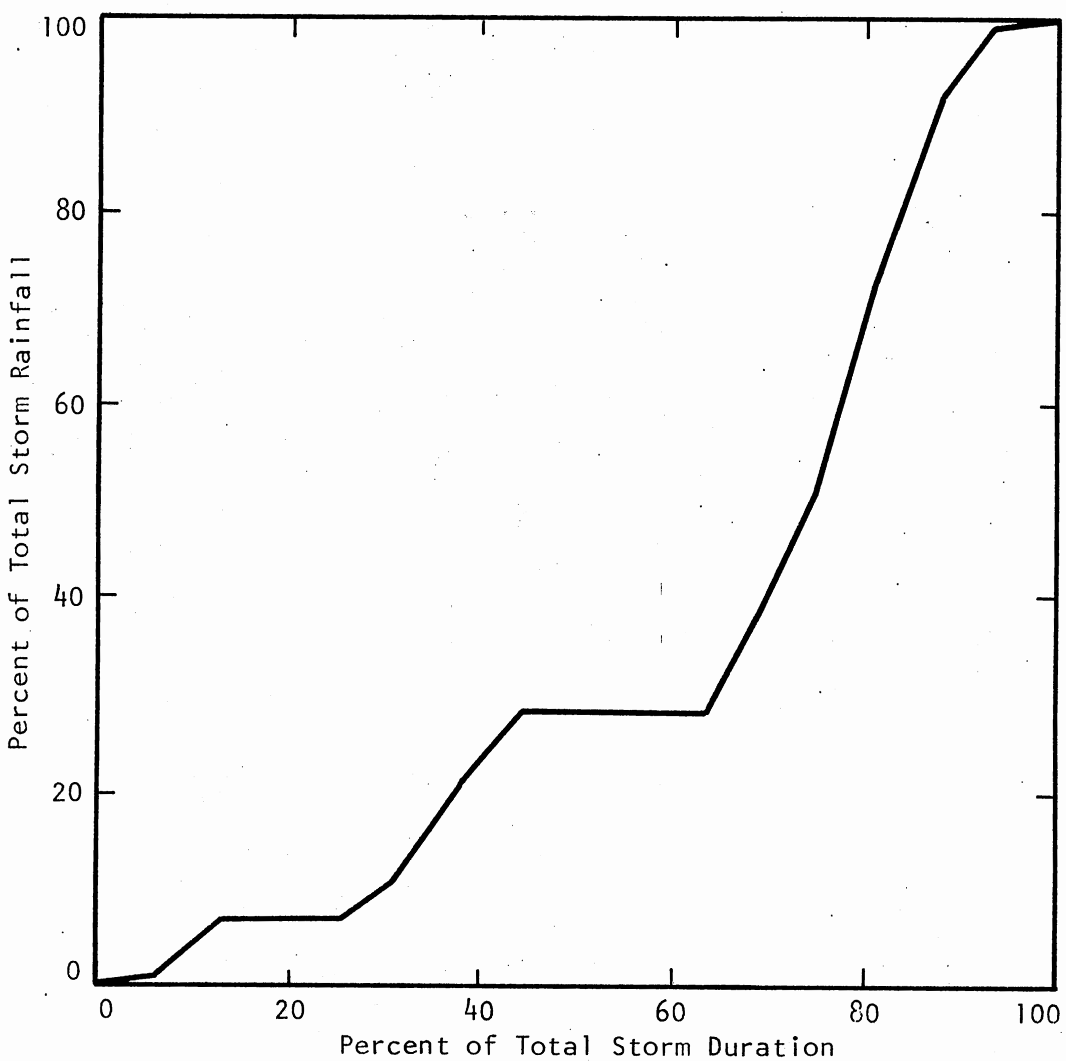
June 20, 1979
Gage No. 12
E. 71st and S. 73rd E. Ave.
Duration 3.25 hrs
Total Rainfall 5.85 in.

Figure 14. Rainfall Pattern for June 30, 1979, Gage No. 12
(100-Year+ Frequency)



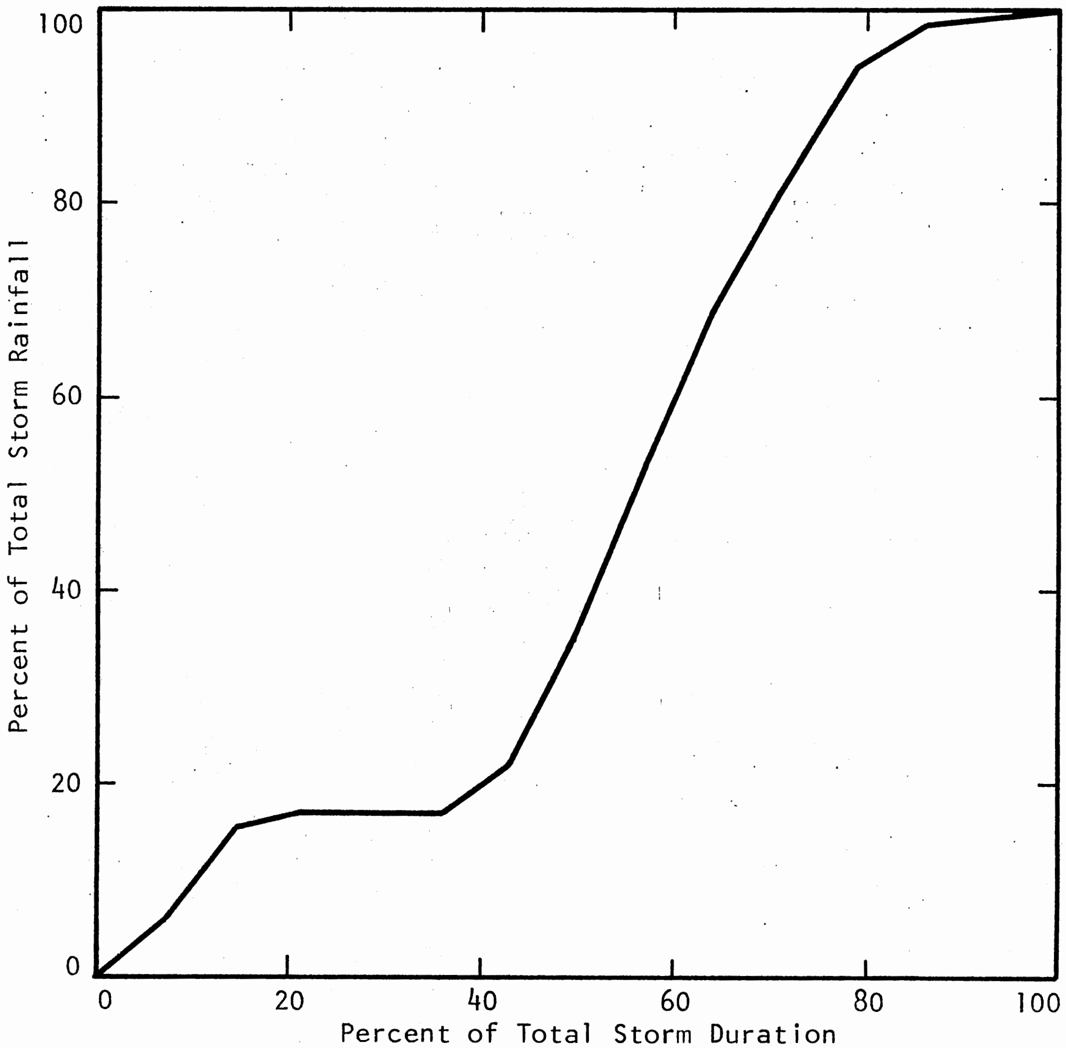
May 9, 1970
Gage No. 5
61st and Mingo
Duration 4.25 hrs
Total Rainfall 5.55 in.

Figure 15. Rainfall Pattern for May 9, 1970, Gage No. 5



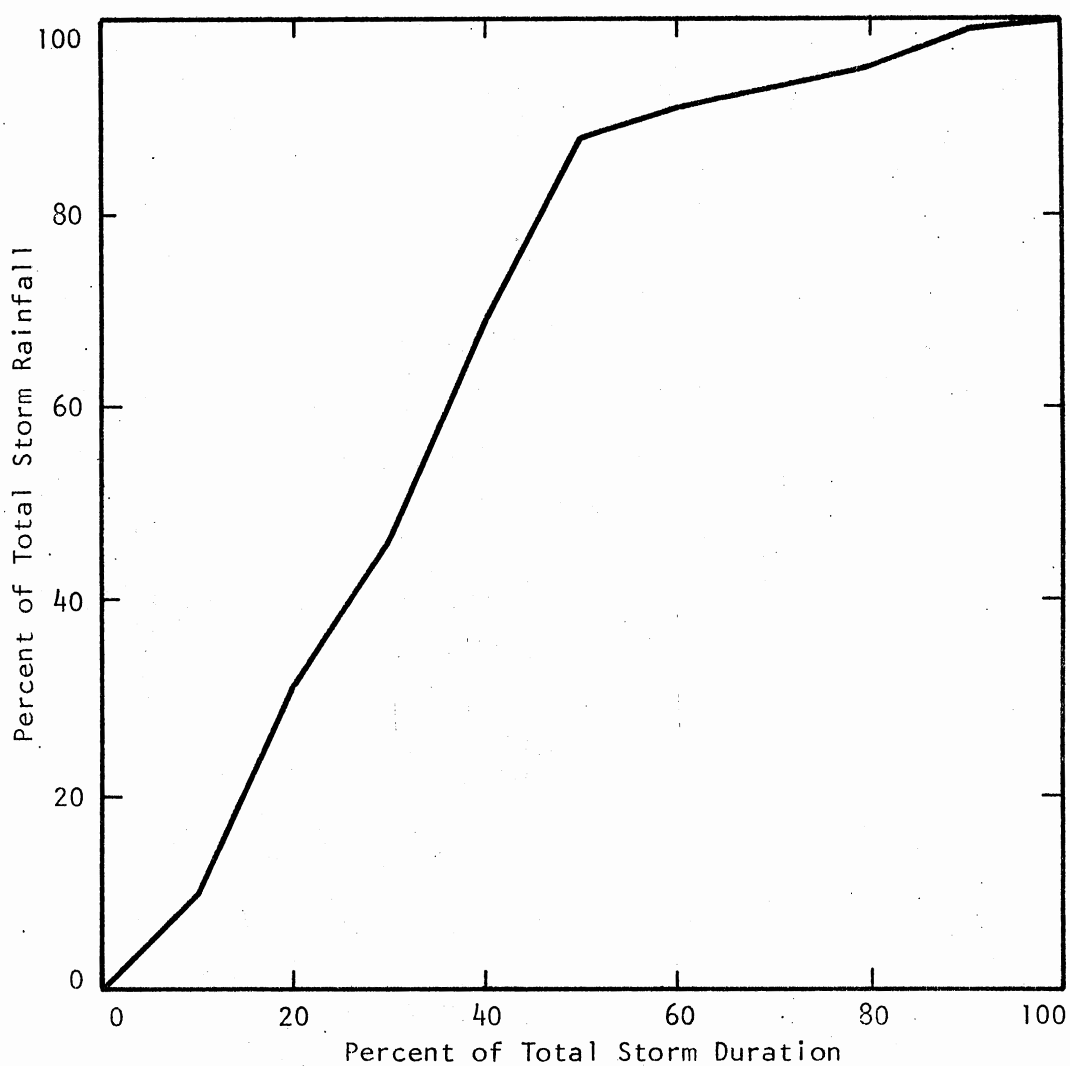
May 30, 1976
Gage No. 3
31st and Urbana
Duration 4 hrs
Total Rainfall 5.35 in.

Figure 16. Rainfall Pattern for May 30, 1976, Gage No. 3



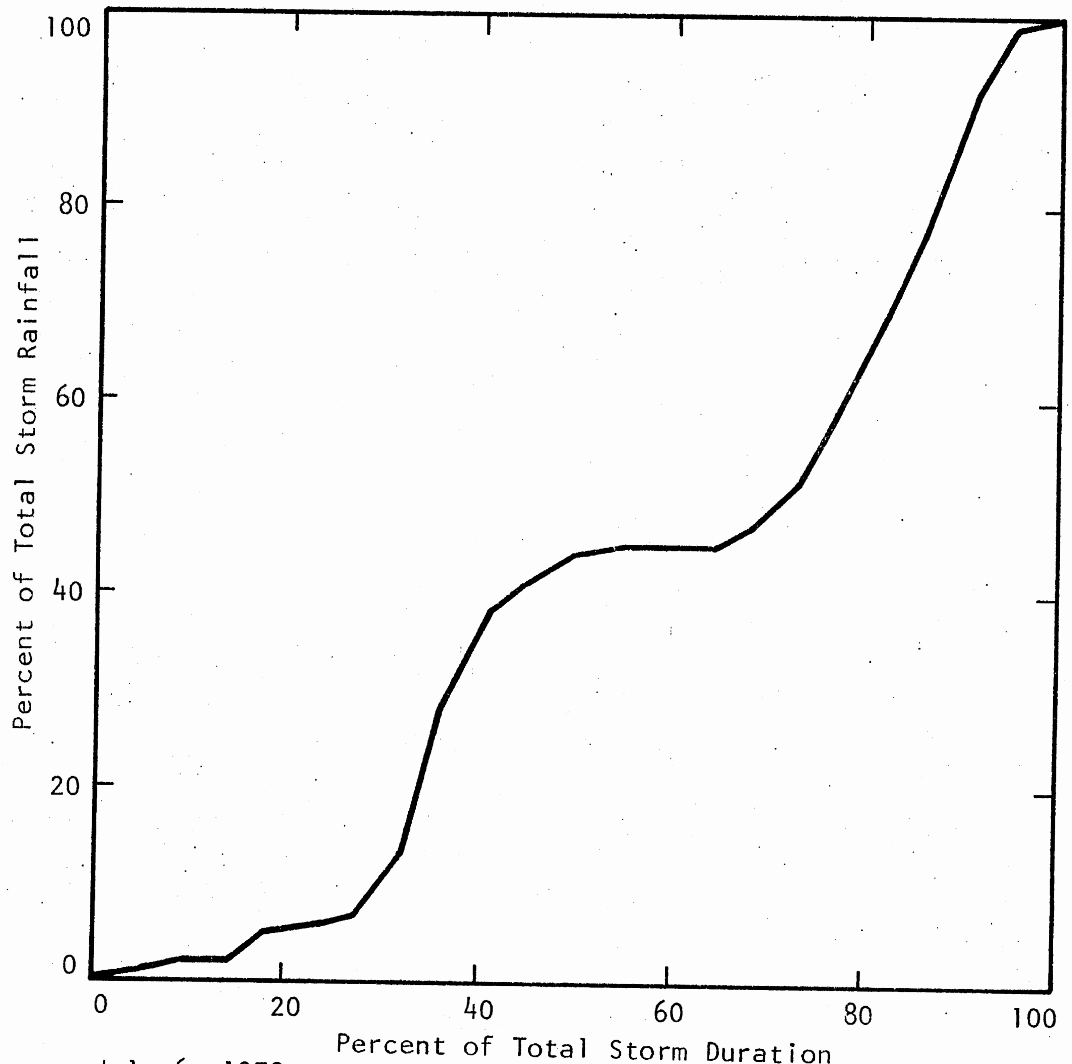
May 30, 1976
Gage No. 5
61st and Mingo
Duration Storm 3.5 hrs
Total Rainfall 7.15 in.

Figure 17. Rainfall Pattern for May 30, 1976, Gage No. 5



June 23, 1979
Gage No. 13
N. 25 W. Ave. and Newton
Duration 2.50 hrs
Total Rainfall 5.15 in.

Figure 18. Rainfall Pattern for June 23, 1979, Gage No. 3



July 6, 1979
Gage No. 14
56th and Harvard
Duration 5.5 hrs
Total Rainfall 5.77 in.

Figure 19. Rainfall Pattern for July 6, 1979, Gage No. 14

VITA

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